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The Panama Crane Contract

By F. H. Cooke,* Civil Engineer, U. S. Navy

SYNOPSIS—Review of available types of crane for the 280-ton Panama service. Revolving-tower type with pivoted jib was found best. Bids asked on general requirements, to bidders' designs, resulted in one American and three foreign designs. Deutsche Maschinenfabrik A. G., of Duisbury, Germany, bid the lowest price and quickest completion. Its price was lowest, even after adding 45% duty. German bid accepted.

Inasmuch as public attention has been drawn to the floating cranes being furnished the Panama Canal, by the published reports of the collapse under test load of the jib of the first crane offered for test (described in an article following this one), a summary is here given of the facts surrounding the preparation of the original specifications, the award of contract to the Deutsche Maschinenfabrik A. G., and subsequent operations. Early in 1912 the Isthmian Canal Commission decided that two very powerful floating cranes would be necessary as part of the equipment of the Panama Canal to handle lock and dock gates, furnish wrecking service, handle heavy pieces of freight, or any other work requiring extremely high lifting capacity.

Fig. 1. Panama Canal Floating Crane "Hercules"
(Carrying the test load of 326 tons at 22.3 ft. reach, Mar. 18, 1915)

Comparison of Types

In order to determine the type of crane best suited to the service, the available literature on the subject was studied, inspection was made of the principal floating cranes in the United States and Canada, and information was solicited from the manufacturers of large floating cranes. Opinions were also sought from the users of cranes of large capacity.

The investigations showed that the floating cranes of large capacity to be found in use in the harbors and shipyards in the United States can be grouped into three classes—the derrick barge, the Bishop revolving crane, and the bridge type. The derrick-barge class may be described as a stiff-leg derrick mounted upon a barge. Usually, a counterweight is provided in the form of water ballast, so that the derrick may be worked over the sides of the barge. Representative cranes of this type are the 200-ton "Monarch," owned by Merritt & Chapman Derrick & Wrecking Co., New York, and the 120-ton derrick at the Norfolk Navy Yard. These cranes are simple, rugged, dependable tools, but their reach when work-

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ing alongside a ship is greatly limited by the interference of the ship's side with the boom, owing to the low elevation of the step of the boom. Their general utility is further diminished by their inability to deposit bulky loads on their own decks without considerable sacrifice of reach over end.

The best example of the Bishop crane is the "City of New York," built in 1871 for the Department of Docks and Ferries of New York City. It is an improvement upon the original crane designed by W. J. Bishop, built in 1866 and still in use in the yards of the W. & A. Fletcher Co., Hoboken, N. J. Briefly described, this type of crane consists of a pontoon carrying a tower, at the top of which a boom revolves in a horizontal plane. The extension of the boom beyond the pivot serves for counterbalancing, the outer end of this counterbalancing boom being secured by backstays to a carriage which runs on the under side of a circular track about the base of the tower. Slewing of the load is accomplished by moving the carriage, which transmits the motion through the backstays to the counterbalancing boom. Two later cranes of this type (for the Boston Navy Yard and for William Cramp & Sons, Philadelphia) are merely types of the "City of New York." The chief disadvantages of the Bishop crane are the difficulty of slewing with maximum load at end of boom and interference with ship's upper works, etc., because of the considerable area required for slewing the boom.

The most recent development of the floating crane in the United States is found in the so-called bridge type—a cantilever crane mounted upon a pontoon (Fig. 2). The first crane of large capacity of this type to be constructed in the United States was the "Hercules," built by the Brown Hoisting Machinery Co. for the navy, and in use at the Brooklyn Navy Yard. There are also cranes of this type recently built by the Wellman-Seaver-Morgan Co. for the Puget Sound Navy Yard, the Boston Navy Yard, and the Pearl Harbor Naval Station, Hawaii.

This type of crane has the advantage of comparatively low cost, but there are several inherent disadvantages. It is topheavy as to weight and wind pressure, the movable counterweight usually provided would probably capsize the crane by its overturning action in its extreme position at one end of the pontoon should the tackle fail at the opposite end of the cantilever, and the size of the load handled by this type of crane is limited by the space between the legs which support the bridge. The greatest disadvantage is the inability to plumb the lift (except as provided by the trolley) without moving the whole crane, which involves the repeated handling of mooring lines. This rigidity is especially objectionable when working about a ship, on account of the interference of the cantilever with the ship's masts and upper works. In narrow waterways, as, for example, the locks of the Panama Canal, the use of this type becomes virtually impracticable, since it can only be operated end on, with the result that the entire length of the pontoon with the cantilever extension is placed across the channel.

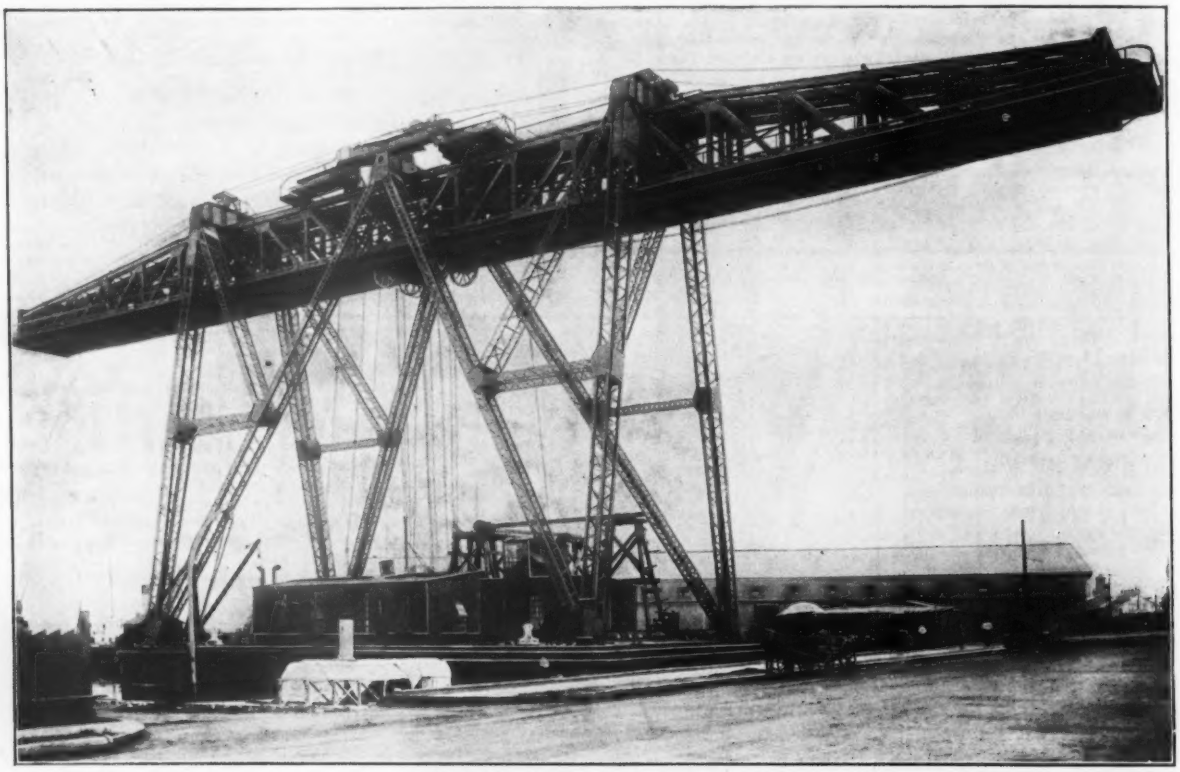

FIG. 2. TYPICAL FLOATING CRANE OF THE BRIDGE TYPE
(150-ton crane of Navy Department at Honolulu, built by Wellman-Seaver-Morgan Co., Cleveland, Ohio)

THE REVOLVING CRANE

The development of the floating crane has proceeded along other lines in Europe, and in its most modern and complete form the floating crane consists essentially of a jib or arm carried by a pontoon, capable of movement

in any vertical plane and of revolution about a vertical axis. It thus differs essentially from the Bishop crane in that the boom which carries the load is hinged to revolve about a horizontal axis, thus greatly reducing interference with ships' upperworks, etc. This type of crane may be termed the revolving type, and is hereinafter so referred to. The development of this type has been carried far beyond that of the Bishop type; it seems safe to say that the highest development of the revolving type of floating crane is also the highest development of floating cranes in general.

The revolving type is inherently more costly than the bridge type, largely on account of the "luffing" of the jib and its revolution about the vertical axis. This was exemplified by proposals received by the U. S. Navy Department in June, 1911, for two floating cranes, each of 168 tons'* capacity at 62 ft. reach over fenders. The specifications under which these proposals were made did not specify the revolving type nor did they exclude that type. Proposals were received from three American firms and from the American representatives of a German firm. The three American firms each bid on a bridge type of crane, the bids ranging from, in round numbers, $628,000 to $579,000 for two cranes. The bid on the revolving type of crane was $1,100,000 for two cranes. This latter bid exceeded the appropriation available and was accordingly dropped from consideration despite the advantages of the revolving type. Contract was finally awarded to the second-low American bidder, who had previously built for the Navy Department a floating crane of the bridge type.

*Throughout this article and the one following, ton means 2000 lb.—EDITOR.

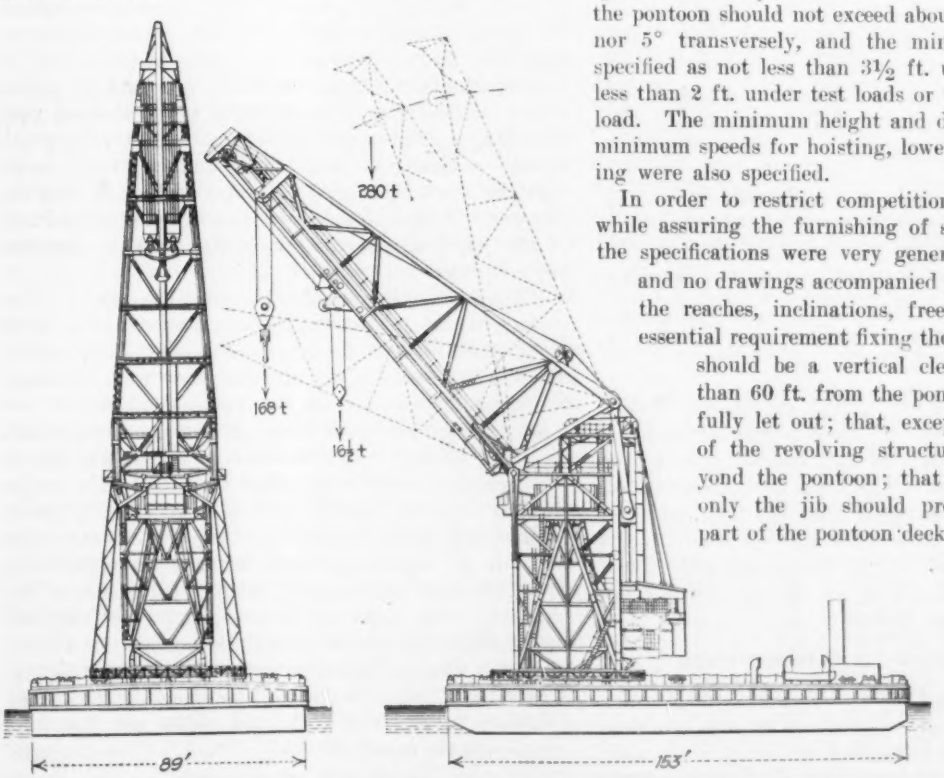

FIG. 3. OUTLINE DRAWING OF PANAMA CANAL 250-TON REVOLVING CRANE

Reproduced from a drawing of the Deutsche Maschinenfabrik A. G., the builders, by courtesy of Neumeyer & Dimond, American agents)

After giving due consideration to the greater cost of the revolving type of crane, the Isthmian Canal Commission decided that the advantages of this type justified the additional cost as compared with the ordinary American bridge type. Since no American firm had built a crane of the type and size desired, a conference was had with an engineer of the German company which had made the bid to the Navy Department in June, 1911, heretofore mentioned, and which later proved to be the successful bidder for the cranes for the Panama Canal. Adding the information so gained to that already in the possession of the Isthmian Canal Commission, a specification was prepared, based on which proposals were opened in the Washington office Jan. 13, 1913.

SPECIFICATION

In brief, this specification required the furnishing of two floating cranes of steel construction throughout, of the revolving type, and of the following capacities and reaches:

280 tons at 22 ft. reach
168 tons at 62 ft. reach
112 tons at 82 ft. reach

The pontoon was specified to be not less than 80 nor more than 100 ft. wide, not less than 130 ft. long, and not less than 14 ft. deep. The restriction as to width was on account of the clear width of 110 ft. existing in the locks of the Panama Canal. The high lifting capacity of 280 tons at the short reach of 22 ft. was largely to enable both cranes working together to unstep an injured gate leaf, this increased lifting capacity being gained at comparatively small increase in cost. The specifications required that the maximum inclination of the pontoon should not exceed about 2½° longitudinally nor 5° transversely, and the minimum freeboard was specified as not less than 3½ ft. under rated loads nor less than 2 ft. under test loads or with 336 tons of deck load. The minimum height and depth of hoist and the minimum speeds for hoisting, lowering, luffing and slewing were also specified.

In order to restrict competition to the least extent, while assuring the furnishing of satisfactory apparatus, the specifications were very general as regards design, and no drawings accompanied them. In addition to the reaches, inclinations, freeboards, etc., the only essential requirement fixing the design was that there should be a vertical clear height of not less than 60 ft. from the pontoon deck to jib when fully let out; that, excepting the jib, no part of the revolving structure should project beyond the pontoon; that of the superstructure only the jib should project over a specified part of the pontoon deck intended for the deck load, under any condition, and that the jib should be able to handle rated loads over two sides and one end of the pontoon, and should further be able to handle these loads to and from the deck of the pontoon.

The circular stated that while bids were desired as nearly as possible in accordance with the specified requirements, the Isthmian Canal Commission reserved the right to give consideration to, and to make award on, bids embodying changes in detail if such changes were considered by the Commission to be advantageous. The circular also stated that in making award the experience, reputation and facilities of the bidders would be taken into account.

Four Bids

Four proposals were opened by the Washington office Jan. 13, 1913—one American, one English, one German, and one Dutch, as follows:

German	2 cranes.....	$820,350	Time 580 days
	1 crane only....	420,175	Time 550 days
Dutch	2 cranes.....	$920,000	.....
	1 crane.....	460,000	.....
English	2 cranes.....	£230,000 (approximately $1,150,000)	Time 548 days
	1 crane only....	£115,250 (approximately $576,250)	Time 548 days
American 2	cranes.....	$1,450,000	Time 690 days
	1 crane only....	745,000	Time 690 days

The American bid was made by the Wellman-Seaver-Morgan Co., Cleveland, Ohio, the successful bidder for the Navy Department cranes hereinbefore referred to.

The proposals were referred to the Isthmus for recommendation as to award, and a committee was appointed to canvass them. Since there was but one American bid of the four bids received, and since this was considerably higher than any of the foreign proposals, it became necessary to consider the provisions of the Act of Congress of June 25, 1906:

> That purchase of material and equipment for use in the construction of the Panama Canal shall be restricted to articles of domestic production and manufacture from the lowest responsible bidder unless the President shall in any case deem the bids or tenders therefor to be extortionate or unreasonable.

Under date of Mar. 2, 1909, this joint resolution was interpreted by the President, as far as it refers to bids on foreign material, as follows:

> The following Executive instructions in furtherance of joint resolution No. 35 of June 25, 1906 (34 Stat. L. 835), will hereafter control in comparing bids for material to be used in connection with the construction of the Panama Canal and in determining awards thereon.
>
> When bids submitted on material of foreign production are lower than on material of domestic production and manufacture, preference will be given to the article of domestic production and manufacture unless the bid for the latter exceeds the former by an amount equal to the duty of the imported article.

The duty was considered in this case to be 45%.

The Dutch bid was informal and was not considered. The English bid was dropped from consideration by the application of the Act of Congress above referred to. The award then lay between the American bid and the German bid, the latter, though increased by the 45% above referred to, being still considerably lower than the American bid, while the German time of delivery was 110 days less than the American.

German and American Bids Compared

The German bidder was the Deutsche Maschinenfabrik A. G., of Duisburg, Germany. This is a combination of several German firms, and had built at the date of opening the proposals more than 30 floating cranes of the revolving type, of which ten or more were of 100-ton capacity and over. The cranes proposed to be furnished by this firm consist essentially of a fixed tower mounted on a pontoon surrounded by a revolving framed structure which in turn carries the jib. The entire weight of the superstructure and hook load is carried by a bearing at the top of the fixed tower, and the overturning moment of the weights and loads is resisted by a horizontal force at the top of the fixed tower and an equal but opposite horizontal force at the foot thereof. This design reduces uncertainty as to stresses to a minimum.* The German pontoon is framed with longitudinal and transverse plate-girders that subdivide it into compartments. The only part of the crane structure proper or operating machinery which occupies the interior of the pontoon is the four feet of the tower, riveted to two longitudinal girders, and the boiler room and electric generating room, which occupy the after part of the pontoon. The entire pontoon is an efficient box girder for carrying the stresses.

The design offered by the American bidder is materially different from that offered by either the German or the English bidder, and also different from the illustrations of other cranes built by the Dutch bidder as appearing in the technical press (these illustrations show a type of crane essentially the same in principle as that offered by both the German and the English bidder). In the American design the weight of the superstructure and the hook load are carried on a circular turntable supported but a short distance above the floor of the pontoon; the overturning moment is also resisted by vertical forces acting on this turntable. The lower part of the revolving structure contains several multiple-web plate-girders of considerable depth, and it is necessary to form a very large hole in the deck of the pontoon for the passage of the revolving structure. That part of the revolving structure visible above the deck also contains several deep plate-girders. The jib is hinged comparatively close to the deck and the luffing action is effected by screw spindles whose working position ranges from approximately horizontal to approximately vertical. In this connection it is to be noted that the screw spindles for similar service in both the German and English designs are vertical for all positions of the jib, thus obviating necessity for their carrying any but direct and torsional stresses.

While complying absolutely with the specified requirements as to capacity, reach, freeboard, etc., the design submitted by the American bidder was obviously much less economical in material than that submitted by either the German or the English bidder. It was a novel design so far as concerns floating cranes of the required capacity, while the German design was but an adaptation to meet the specified conditions of a design of crane which had been successfully used in many places. Interruption of the continuity of the pontoon structure, not only as regards its deck, but also as regards the structural framing, necessitated by the location of the turntable in the American design, is a further very real disadvantage, not only as regards economical use of material, but also as regards continued water-tightness. It is here to be noted that notwithstanding the very great difference in prices bid, *the unit-prices per ton were very nearly the same.*

*The English design was along identical lines so far as essentials are concerned, the principal difference being that the weights were carried at the foot of the mast rather than at its top.

Leaving out of consideration the lower price and shorter time bid by the German firm, the advisability of investing nearly $1,500,000 in apparatus of a design heretofore untried was considered very questionable, particularly as it appeared that a considerable part of the money which would have to be expended would be for material which could be eliminated by a more economical design.

Of the two bids the American was considerably more complete than the German, and fully met the specified requirements as to capacity, reach, etc.; in some cases exceeding these requirements. The German bid did not fully meet the specified requirements as to reaches in all cases, being under in some and over in others. Furthermore, in many instances, this bid did not show the design of portions of the crane actually bid upon, but it did show in these instances similar parts of similar cranes either already built by them and in successful use or in construction at the time, and while, technically, this might be considered as noncompliance with the conditions of advertisement, it was actually more favorable as showing something actually built rather than a part of an untried design. The following tables show the comparison between the specified capacities and reaches and those included in the German bid:

REACH OVER END, FEET

	112 Tons	168 Tons	280 Tons
Specified	82.0	62.0	22.0
Proposed	81.0	59.3	21.4

REACH OVER SIDE, FEET

Specified	82.0	62.0	22.0
Proposed	82.4	63.6	24.2

It will be seen that the reaches over end are slightly less than those specified, while those over side are slightly in excess. The contractor in his proposal called attention to the discrepancy and stated that "at much increased cost the out-reaches could be increased over the end." After the opening of the bids the contractor quoted a price of $13,400 per crane to widen the pontoon from 88 to 89 ft. and to fulfill exactly the requirements of the circular. This offer was not accepted, as the extra expenditure did not seem justified, since the reaches, as proposed, met the requirements of the Panama Canal and were satisfactory, inasmuch as in the majority of cases the crane would lie side on to the ship or wharf from which the loads were lifted, in which case the full advantage of the slightly excess reaches over the side would be obtained.

The German proposal included several alternates as regards the power plant, and the contract, as awarded, was based on the adoption of one of these alternates which increased the base price bid. It is interesting at this point to compare the cost of the German bid increased to gain the specified reaches in every particular plus the accepted alternate referred to above plus certain other minor additions, without and with the addition of the 45% duty, on the one hand, with the American bid on the other, and this comparison is as shown in the table in the next column.

It is thus seen that without the addition of duty there was a balance of approximately $580,000, or enough to purchase a third crane, in favor of the German proposal, and with the addition of duty there was a balance of approximately $190,000 in his favor.

German Bid Accepted

In the face of these facts the board did not feel itself justified in recommending award to the American bidder, and accordingly recommended award to the Deutsche Maschinenfabrik A. G., of Duisburg, Germany. This recommendation was approved by higher authority on the Isthmus and in Washington, and award was made accordingly, contract being entered into with the Deutsche Maschinenfabrik A. G. as of date Apr. 21, 1913.

BIDS COMPARED, DUTY INCLUDED

	German	American
Base bid	$820,350	
Bidder's alternate proposal No. 1	7,200	
Modification in motors	9,350	
Limit switches	600	
Original contract price	$837,500	
To meet the specified requirements as to reaches	26,800*	
To make the interior bulkheads water-tight	5,000*	
To carry the dead weight of the equalizer bar	2,000*	
Total to meet the specified requirements	$871,300	$1,450,000
Plus 45% duty	392,085	
Total	$1,263,385	$1,450,000

*Of the three items marked the first two were not ordered, as the additional cost did not seem to be justified. The third item ($2000) was provided for by the contractor without additional compensation, as it represented an error in his proposal.

An Assistant Engineer of the Isthmian Canal Commission, especially skilled in mechanical engineering, and thoroughly conversant with the German language, having been educated in Germany, was sent to the contractor's works in Germany as the representative of the Isthmian Canal Commission, and as such was authorized to check and approve all contractor's plans, working drawings, and calculations. He was also in charge of the inspection of all material and workmanship. He remained in Germany during the period of fabrication and until the cranes had started for the Isthmus, and during that time he approved all plans and work. He was assisted by one American structural engineer having a knowledge of the German language, appointed in the United States, who was highly recommended by his previous employer, and who was sent to Germany for this purpose. He was also assisted by a force of German subinspectors employed in Germany. None of the contractor's working drawings were checked or approved either in Washington or on the Isthmus. Some of the inspection was performed by the Germanic Lloyd, including mill inspection of materials, and the inspection and testing of boilers and piping and the inspection of anchors.

On Apr. 26 and May 30, 1914, respectively, the two cranes were dispatched in tow from Emden, Germany, for Cristobal, Canal Zone. They arrived on the Isthmus safely on July 8 and on July 13. Erection of the jibs had been started at Gatun in June.

[*The failure of the jib of the "Ajax" under the acceptance test is described by Mr. Cooke in the article beginning on the next page.*—Editor.]

Bridge Piers Built in 40-Ft. Tide—A steel railway bridge on concrete piers was recently completed over the Avon River, Nova Scotia, for the Canadian Pacific Ry. The Avon flows into Minas Bay, an affluent of the Bay of Fundy, which accounts for the fact that tides in the river reach 35 to 40 ft. The bridge is 1200 ft. long c. to c. of abutments, with nine concrete piers all on rock or hardpan, 35 ft. below river bed, except one which is carried on piles. The rock is overlaid with 5 ft. of gravel covered with 15 ft. of shifting sand. The current reaches as high as 8 mi. per hr. Foundation work was extremely difficult, and had to be done at low tide—allowing only 3½ hr. per day for a shift. The bridge was built in two years by the Nova Scotia Construction Co., Ltd., of Sidney, N. S., with A. S. Kennedy, Construction Superintendent.

Failure of Panama Crane "Ajax"

By F. H. Cooke,* Civil Engineer, U. S. Navy

SYNOPSIS—The first of the two cranes was tested under 112 tons at 82-ft. reach, then under 20% overload, which it carried; but on starting the luffing motors rivet-heads came down and the jib fell, its heel strut and some other members crushed. The heel strut and web members of the jib are under maximum stress in the 82-ft.-reach condition. The heel strut is held to be the point of failure. Deficient latticing, with one-rivet attachment (see Fig. 3), and inadequate provision of diaphragms are concluded by the Canal to be the cause. The heel strut and some other members of the jib of the "Hercules" were reinforced, and the crane went through its tests successfully. A new jib is being built for the "Ajax."

When the two 280-ton floating cranes built by the Deutsche Maschinenfabrik A. G. were shipped from Germany they were at the following stage: The pontoon was complete with tower, the boiler and generating plant was in the pontoon ready for service, and the revolving superstructure was complete except for the jib, counterweight arm, spindles and carriage, winch house and hoisting machinery. Spindles and carriage were lashed to the deck of the pontoon. All other parts not erected were shipped knocked-down, by steamer from Antwerp to the Isthmus.

The jibs were erected on the east wall of the middle chamber at Gatun Locks. As one was completed, the crane to which it belonged was brought into the lock chamber and the water level regulated until the correct relative height was obtained. The jib was then skidded on rollers until it projected sufficiently beyond the lock wall and was pinned to the superstructure. The connecting links between jib and crosshead were fitted with an extension, and by lowering the crosshead the jib was raised until about the midway position. It was secured in this position, the extension removed, and the connecting links pinned to the jib. With the contractor's consent, the Panama Canal drydocked both cranes in the upper chamber at Gatun and cleaned and painted the underwater portion, the time of delivery being correspondingly extended.

The Panama Canal found it necessary to remove a sunken drill barge which was impeding navigation through the Culebra Cut, and the contractor's representative offered the "Ajax," the first of the cranes to be completed, for this work, prior to test and acceptance, with the understanding that the delivery date on both cranes be extended. The crane removed the two portions of the sunken drill barge, and later, at Gatun Locks on the return trip to Cristobal, transferred three towing locomotives from the east wall to the center wall. The crane performed this work most successfully; it operated easily and well, and no defects in operation or weakness in any part were observed. During the work of removal of the sunken drill barge, the maximum lift was about 150 tons at 18-ft. reach.

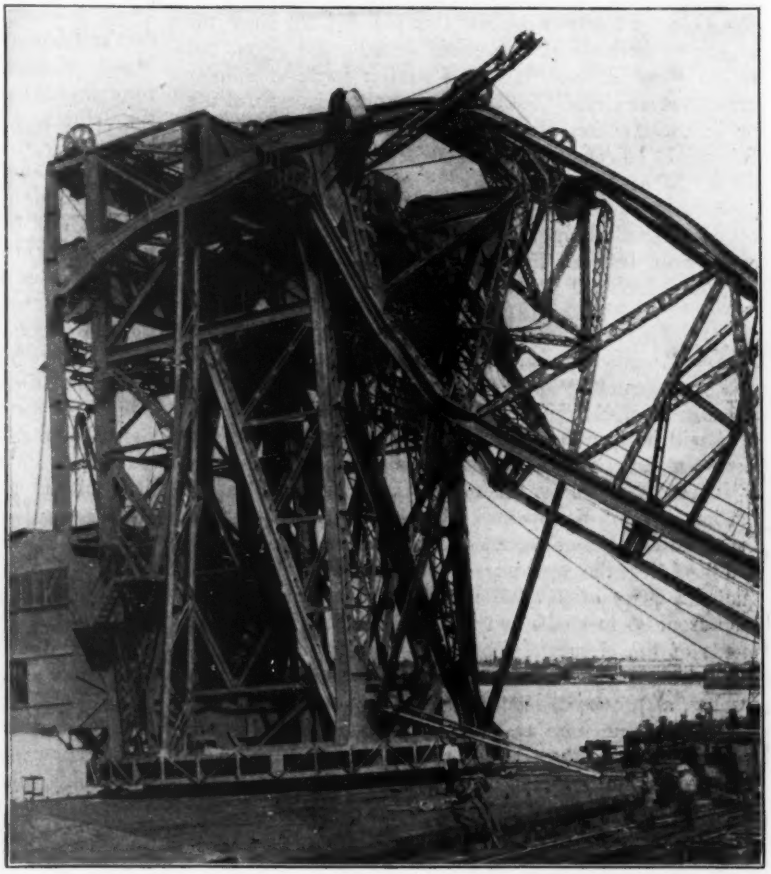

Fig. 1. Right-Hand Side
The Wreck of the Crane "Ajax" During Its
(The crane jib collapsed while under a test load of one-rivet latticing. Both views show the heel strut left-hand strut buckled first)

Test of "Ajax"

The original contract date of completion was Nov. 22, 1914. On Dec. 1, 1914, the contractor offered the crane "Ajax" for the acceptance trials. A test load of 112 tons had been constructed, to be increased by cast-iron blocks and steel rails to 336 tons.

The actual test began on Dec. 3. By the terms of the contract, the contractor was responsible for the conduct of the tests. The crane withstood successfully the 112-ton normal-load test. The hoisting speed, luffing speed

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and slewing speed were measured, and the contract requirements were met. On Monday, Dec. 7, the test load was increased 20%, as specified by the contract for the overload tests, by the addition of steel rail. The 134-ton load was hoisted clear of the wharf and luffed slowly out to the 112-ton reach; that is, to a reach of 81.6 ft. from the face of the fender. No sign of distress was noticed in any member.

Jib Collapses When Luffing Motors Start

It was noted then that the auxiliary hoist trolley was at the inner limit of travel, when according to contract it should be at the tip of the jib for the maximum loads. The jib was to be luffed in so that the trolley could be run out, but the instant that the controller was placed in the luffing position and the brake solenoids were heard as they released the brakes on the drums, rivets were heard snapping aloft and broken rivets began to fall. Then two movements were observed: First, the collapse of the back of the jib and fall of the load, which was only a short distance above the ground; and then, after a pause, the fall of the jib and the recoil of the pontoon.

The four photographs, Figs. 1, 2, 4 and 5, show the appearance of the wrecked crane. In comparison the appearance of the undamaged crane may be seen in the front-page view of the "Hercules" during its test, and in Fig. 4 the "Hercules" before reinforcement of its heel members.

The jib pivot remained intact, and the jib in its fall turned about this pivot. The head of the jib, with the sheaves which support the main load, swung clear of the test load and buried itself about 6 ft. in the ground. The extension of the jib beyond this point, which serves as an attachment for the trolley sheaves, struck the test load and was completely wrecked. The lower chord was bent up at the second panel-point from the pivot, and the top chord was bent down at the first panel-point on the top chord. The top chords were bent down until the upper eyes which hold the connecting-link pins were brought down over the jib-pivot eyes, but back and to the left of them.

Fig. 2. Left-Hand Side
112-Ton Test at Gatun Locks, Dec. 9, 1914
134 tons at 82-ft. reach. The heel strut of the jib crushed by failure of its
completely buckled, but the Panama Canal authorities concluded that the

The heel struts (29, see numbering diagram in Fig. 3) which take the compression between the upper pin and the pivot of the jib were bent outward, sharp bends occurring in both cases near the points of connection, and at these bends the built-up channels forming the strut were crushed together. The sharp bend at the upper connection consisted of a bend in both directions; at the lower connection the strut was doubled upon itself.

Strut 27 on the right side broke clear of the upper chord at the connection and was not crushed; on the left side the connection at top and bottom held, but the strut was crushed. Members 23 and 21 were bent inward. The portal which ties the upper chord together at the

first panel-point and supports the guide sheaves for the main hoist remained in place. The connecting links from jib to crosshead were badly twisted.

The back spindle-carriage track was sprung outward and with it the two spindles. The spindles, however, were found to have suffered no injury. The main hoist blocks and the equalizer bar were injured, more especially the latter, but the block sheaves were intact. Of the ten upper sheaves at the head of the jib, only two were broken. The ladders, walks, railing, etc., were bent, and the trolley track was badly twisted and bent. The jaws of the revolving superstructure, which form the jib pivot, were twisted, and the outer cover-plate was ripped. The inner part of the trolley track penetrated the operator's cab and injured the controllers and instruments. The hoisting rope was kinked in places and rendered unfit for use.

Investigation of the Cause of the Failure

The jib is a framed structure consisting of two trusses, each as shown in the numbering diagram in Fig. 3. The lower chords of the trusses are connected by cross-bracing.

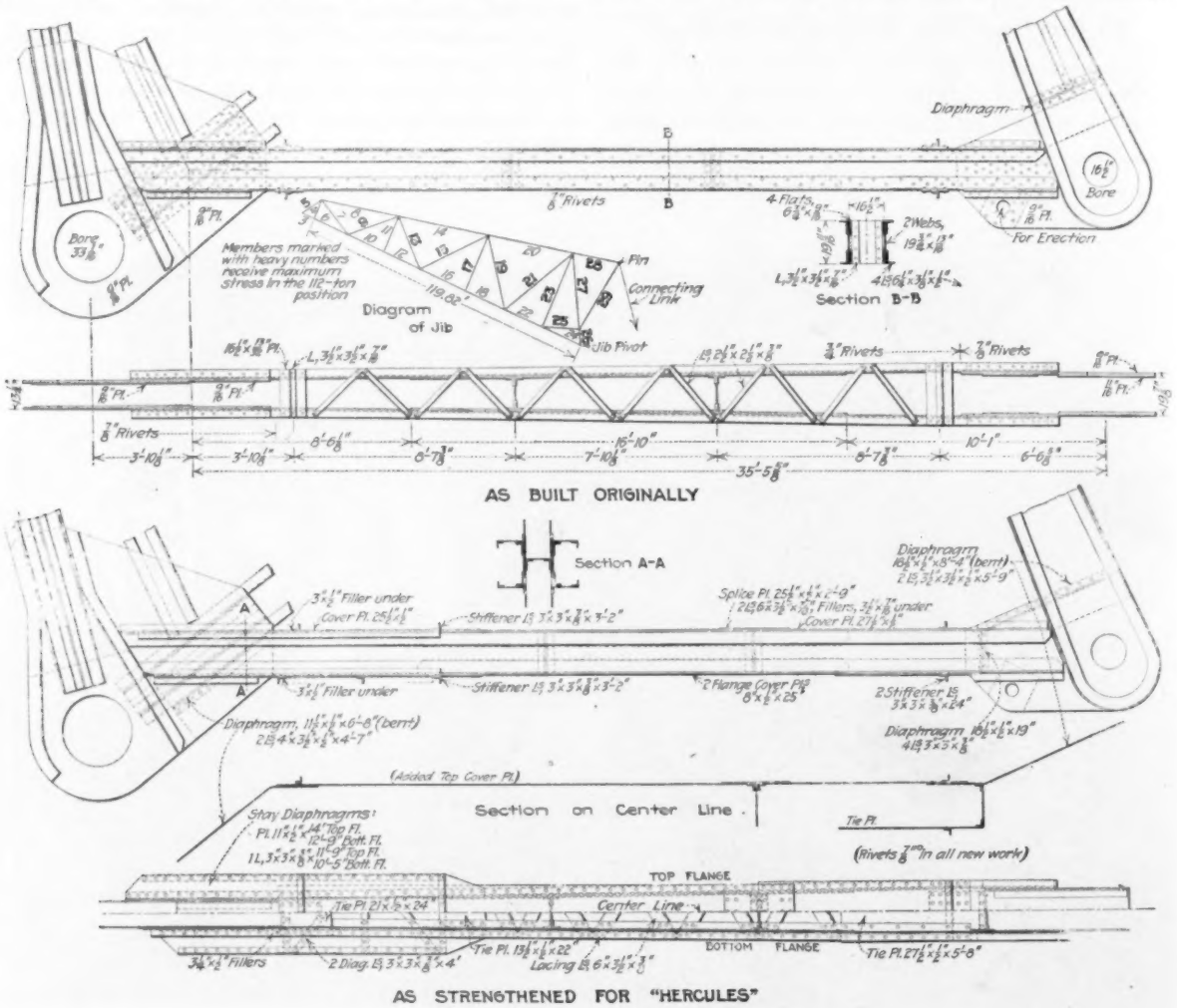


Fig. 3. Details of Heel Strut

(As built for the crane "Ajax," following the original design, and as reinforced for the crane "Hercules." The front-page view shows how the "Hercules" after being fitted with the reinforced heel struts carried its 336-ton test load)

TABLE I—STRESSES IN HEEL-STRUT (MEMBER 29) IN 1000-LB. UNITS IN THE 112-TON POSITION

	1 Condition "A" As estimated by contractor with 112 tons suspended load, 120 ft. radius, 5° inclination, 10 lb. wind.	2 Condition "A" As estimated by Panama Canal with 112 tons suspended load, and reach, inclination, and wind as obtaining at the time of test.	3 Condition "B" As estimated by Panama Canal with 135 tons suspended load, and reach, inclination, and wind as obtaining at the time of test.
Dead-load	168.2	161.0	147.8
Live-load	383.6	378.2	432.3
Wind	9.0	....	0.3
Total	560.8	539.2	580.4

TABLE II—ELEMENTS AND UNIT-STRESSES

(In inches and lb. per sq.in.)

	Lower End	Middle	Upper End
Area	48.1	62.8	48.1
I_x	2,656	4,898	5,521
I_y	2,049	2,727	2,049
Least radius of gyration $= 35.4$ ft.	6.53	6.58	6.53
$\frac{l}{r}$	65.2	64.8	65.2
Condition A Permissible unit-stress, $14000 - 70\frac{l}{r}$	9,440	9,460	9,440
Actual unit-stress	11,220	8,580	11,220
Condition B Permissible unit-stress, $18000 - 90\frac{l}{r}$	12,130	12,170	12,130
Actual unit-stress	12,070	9,240	12,070

but the plane of the upper chords is devoid of cross-bracing except for a portal at the first panel-point from the inner end. There are cross-frames in the planes of members 4, 11, 17 and 23. Cross-bracing in the plane of the upper chords was purposely omitted to secure flexibility—if the jib were a rigid structure and the alignment not perfect, a condition could be conceived in which one truss and one connecting link would be forced to carry an excessive percentage of the whole load. There is no cross-bracing in the plane of member 29, because of the necessity for allowing that portion of the jib to pass on either side of the girder that supports the "bell," or revolving structure below the jib, which carries the winch house, operator's cab, and other necessary equipment.

The point at which the connecting links act, the intersection of members 28 and 29, is therefore not braced against lateral stresses; bracing, although desirable, not being thought necessary by the designer. The pivots of the jib are provided with spherical bearings to enable them to adjust themselves should both pins not be on the same axis. The designer of the crane frankly endeavored to keep down the weight of the jib without exceeding the specified unit-stresses, since the weight of this portion of the crane affects the whole structure.

The consensus of authoritative opinion from the viewpoint of the Panama Canal is that member 29 was the first to fail, and that this failure was due, not to deficiency in sectional area, but to *insufficient provision of latticing.*

FIG. 4. CRANE "HERCULES" CLEARING THE JIB OF THE "AJAX," DEC. 14, 1914

(Jib of "Hercules" still in its original condition, unreinforced. The heel strut and nearby compression members of the "Hercules" were later strengthened to correct the weakness shown by the "Ajax")

batten plates, etc., to tie the two parts of the member together.

Paragraph 116 of the specifications refers to compression members:

> In compression members the metal shall be disposed as far as practicable from the axis. No compression members shall have a length exceeding 40 diameters, nor a value of $\frac{l}{r}$ exceeding 100 for main members, nor 120 for wind bracing. Lattice bars, stay plates, diaphragms, etc., together with rivets, shall be generously used to insure the mutual support of the various parts forming the cross-section.

Reference to Fig. 3 will show that member 29 consists of halves, each half being built up of a plate and two angles, two web-plates being added for a portion of the length of the member. Fig. 3 further shows that each individual half is relatively weak as regards transverse stiffness, that the end batten-plates are short, and that the latticing consists of 2½x2½x⅜-in. angles at about 45° inclination, *with one ¾-in. rivet at each end.* Inspection of the photograph Fig. 5 shows that the latticing was in most cases not bent but the rivets sheared off—almost as neatly as if by a shearing machine.

The steel was apparently of excellent quality, this being evidenced by the bending which it withstood without fracture. All of the rivet holes in the jib were either subpunched and reamed or drilled in the solid, and the question of workmanship evidently did not enter into the failure. The material was inspected in Germany, and the mill tests on material later worked into member No. 29 gave results shown in the first part of the table below.

TESTS OF MATERIAL IN MEMBER 29

1. Mill Tests.

Section	Ultimate Strength, Lb. per Sq.In.	Elongation, %	Cold Bending, Quench Bending
Plate	59,000	30	Good
Angle	55,000	29	Good
Flat	54,000	27	Good

2. Tests of Specimens from Finished Member.

Samples Marked	Elastic Limit, Lb. per Sq.In.	Ultimate Strength, Lb. per Sq.In.	Elongation, % in 8 In.
1—Angle	32,600	50,400	30.0
2—Angle	33,000	50,600	28.8
3—Flat	32,950	49,570	27.4
4—Flat	31,830	49,100	29.2
5—Web	35,020	54,000	31.4
6—Web	35,570	54,180	30.2

The requirements for the structural steel were as follows:

> Ultimate tensile strength in the direction of rolling, between 52,500 and 62,500 lb. per sq.in., with a minimum elongation of 20%.
>
> Cold and Quench Bending Test—To be bent in the direction of rolling 180° around a diameter equal to the thickness of the testpiece, without sign of fracture on the outside of the bent portion.

Six specimens cut from member 29 were tested at the Balboa shops of the Panama Canal with results also shown in the table. The fractures were in every case "fine silky." The material was also subjected to cold-bending and quench-bending tests, which showed that the material conformed to the contract requirements in this respect.

It will be noted that the ultimate tensile strengths developed by tests on the Isthmus were less than those shown by the tests in Germany. The reason for this disagreement has not yet been ascertained.

Stresses

In Fig. 3 are tabulated the computed stresses in member 29 in the 112-ton position: First, as estimated by the contractor for 112-ton suspended load; second, as computed by the Panama Canal for 112-ton suspended load; and third, as computed by the Panama Canal for 134-ton test suspended load. The reason for the differences between the first two estimates is that the contractor estimated on the basis of a pontoon inclination of 5°, while the estimate made by the Panama Canal is on the basis

FIG. 5. LEFT-HAND HEEL STRUT (MEMBER 29) OF JIB OF THE "AJAX"

(Photographed Jan. 15, 1915. The weakness of the latticing of this member, shown very plainly by the sheared-off rivets—one rivet in each end of the 2½x2½x⅜-in. lattice angles—is believed to be the cause of the failure of the jib)

of the inclination existing at the time the crane was tested. Similarly, actual conditions have been used in estimating the stress at the time of failure; that is, actual inclination and reach, actual wind, and dead-load of trolley at inboard end of jib.

The elements of member 29 considered as a whole and the permissible and actual unit-stresses theoretically existing are also given in the table in Fig. 3. The area at the middle of the member is about 30% in excess of that at the ends, and the unit-stresses at the middle are well within the specified limits, while at the ends the unit-stresses in Condition *A* exceed the permissible limits by about 100 lb. per sq.in.*

Each half of member No. 29 was further investigated as regards its strength as a short column of length represented by the distance between ends of lattice bars, with the result that the maximum value of $\frac{l}{r}$ is 58.7 for the end sections and 42.8 for the middle section. These values are in each case less than for the member considered as a whole, and therefore the member is theoretically safe.

It is conceivable that, owing to the unstayed condition of the intersection of member 29 with member 28, a component of the dead-weight could cause bending stresses at the foot of member 29 due to the longitudinal inclination of the pontoon which existed at the time of failure. This inclination was but slight, however, not exceeding about 2%, and since the dead-load at the point in question is but 8.2 tons the additional stress at the bottom of member 29 cannot be large, especially when the action of the portal in member 28 and the bracing in the plane of member 23 are taken into account.

It appears, therefore, that the theoretical stress existing in member 29 at the time of failure implied a factor-of-safety of at least 2, figured on the elastic limit of the material. No evidence was found pointing to local defects in material or workmanship.

There was a crunching sound heard at the jib pins during the tests preceding the one in which failure occurred, but examination of these pins after the collapse showed that they were unscored and that the crunching probably arose from the movement of the jib on the spherical bearing surrounding the jib pin, which does not affect the force necessary to luff in the jib.

An excessive stress could have been caused if the main hoist drums had not been free to move, but immediately after the accident the magnetic brakes on the main drums were examined and found free.

Since member 29 failed just as the jib commenced to be luffed in, which luffing in was necessary because the dead-load of the trolley was not at the upper end of the jib, and the existing inclination would be exceeded with the trolley run out without luffing the jib in, it is possible that vibrations were set up in member 29, which is one of the two members meeting at the point where the luffing-in pull is applied, with the result that the rivets connecting the halves of the member, already near their limit of resistance, were finally sheared, causing the member to consist of two weak halves, with the result that they bowed outward. This action, once started, would continue, the adjoining top-chord member would commence to bend, and the collapse of the entire inboard end of the jib is then inevitable.

*The original specifications gave two sets of permissible unit-stresses; one set, known as Condition A, being for loads within the rated capacity with wind not exceeding 10 lb. per sq.ft.; the other set, known as Condition B, being for the crane under conditions of test load with wind pressure assumed at 5 lb. per sq.ft., and for the unloaded crane with wind pressure at 40 lb. per sq.ft. Since the test loads were but 20% in excess of the capacity loads, while the permissible unit-stresses were nearly 30% in excess of the working unit-stresses, the contractor did not consider it necessary to make complete calculations for Condition B, confining these calculations to cases where wind pressure was the determining feature.

Summary of Facts and Conclusion

The opinion that the collapse of the jib was due to initial failure of member 29 and that this initial failure was due to insufficient provision of latticing, battens, etc., is based on the foregoing facts, which may be summarized as follows:

1. There is no indication of faulty material or workmanship at any point of the jib.
2. There are no indications that either during manufacture, erection or use the crane suffered damage or was subjected to loads beyond its capacity.
3. The hoisting drums were free, and no defect could be found in the luffing machinery by running the spindle carriage up and down after the wreck had been cleared away.
4. Rivets snapped and fell just prior to the collapse of the jib. An examination of the wrecked jib showed that the rivets connecting the halves of member 29 were sheared, while the maximum unit-stress in the member was well within the elastic limit. Similar shearing of rivets was not observed in other members.

Operations Subsequent to Failure of the "Ajax"

The contractor did not share the opinion of the Panama Canal that the failure was due to insufficient lattice bars, batten plates, etc., and he expressed such confidence in his design that on Dec. 10, 1914, he offered to subject the "Hercules" to exactly the same conditions as those under which the jib of the "Ajax" collapsed. In view of the circumstances the Panama Canal was unwilling to permit this to be done without reinforcing certain members of the jib. This point of view was also taken by the underwriters.

The reinforcement of the jib of the "Hercules" was completed Jan. 27, 1915; it was executed in accordance with drawings made by the Panama Canal and was confined to members 27, 28 and 29. Member 27 was given additional tie-plates; in member 28 the internal system of latticing was replaced by an internal diaphragm plate. In member 29 the top latticing on one side was replaced by a continuous cover plate, on the bottom the 2½-in. angle latticing was replaced by 6x3½-in. angle latticing with four rivets at each end instead of one, and additional diaphragms and batten plates were provided. This reinforcement is shown in Fig. 3.

The contractor ordered a complete new jib for the "Ajax" to be fabricated in Germany, incorporating substantially the reinforcement above described for the "Hercules" plus minor additions to other members, to make the details conform more nearly to American practice.

After the completion of the reinforcement of the jib of the "Hercules," the crane was in the service of the Panama Canal for about one month prior to acceptance tests. These acceptance tests were completed Mar. 27, 1915, without mishap. The crane met the contract requirements fully as regards capacity, reach, and speed of all motions.

Drilling 30-In. Wells for Irrigation

A prominent feature in the agricultural development of the Southwest is the utilization of underground water supplies for irrigation, not only to save the crops in case of long periods of drought but also to supplement the ordinary rainfall. The water is usually raised by vertical-shaft centrifugal pumps, suspended in the wells. A plant of this type at Garden City, Kan., was described in ENGINEERING NEWS of Oct. 8, 1914. For efficient service these pumps need to be larger than those for domestic water-supply, and the wells are usually drilled to a diameter of 30 in., with a 24- or 26-in. lining.

This system of irrigation has been introduced to a large extent in the northwestern section of Texas, and was commenced in 1910, which was a year of very light rainfall. The first well was sunk by D. L. McDonald, of Hereford, Tex., who has since sunk a large number throughout this section. The following particulars of the work are compiled in part from a statement furnished by him.

FIG. 1. DRILL-BIT

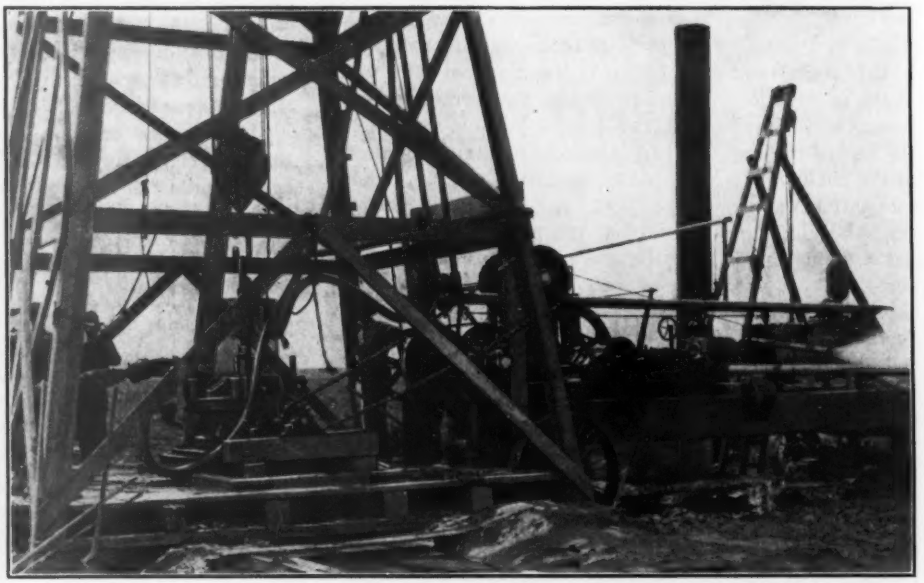


FIG. 2. RIG FOR DRILLING 30-IN. IRRIGATION WELLS, 125 TO 250 FT. DEEP

In this part of Texas the average annual rainfall is about 22 in., and most of this falls during the growing season, but with the high altitude (3800 ft.) and the large amount of sunshine the evaporation is so great that additional water is needed for the crops, even in ordinary years.

The wells are drilled to a depth of about 150 ft. through strata averaging about as follows: 4 ft. of soil, 8 ft. of gray clay, 32 ft. of yellow clay, and 15 ft. of a decomposed limestone that is stratified and heavily charged with water. Below this will be alternating strata (4 to 10 ft. thick) of fine soft sand and very soft sand rock (all water-bearing) extending to a total depth of about 252 ft.

A well to produce 1200 gal. per min. should be drilled to a depth of about 125 ft., and in this formation it would strike water at about 44 ft. The pump should be so placed as to have about 40 ft. of submersion, with 40 ft. of suction pipe below it. If the length of suction pipe is much less than this, the large delivery of the pump may draw down the water level to within a short distance above the end of the pipe, in which case there will be a considerable intake of air, reducing the efficiency.

Coarse screens should be placed at the several strata of soft sand. The screens will allow the sand to pass, and after about 10 days' pumping the water will be clear and there will be a steady delivery of about 1200 gal. per min., with less draw-down than while delivering 600 gal. when pumping is commenced. Experience has shown that if fine screens are used to hold back the sand the water will be held back also, the sand packing tight against the screen. As large quantities of water are required for irrigating, coarse screens must be used and the sand removed, thus forming cavities which afford a large surface for the inflow from the water-bearing strata.

The pumps are usually of the centrifugal "turbine" type, suspended within the well. They are driven by oil engines, which give power at very low cost, so that profits can be made even when irrigating ordinary farm crops.

DRILLING THE WELLS

The method of making these large 30-in. wells is to drill first an 8-in. hole to determine the formation. An accurate log of the well is kept in order to govern the location of the pump and the screens. Following this, a 16-in. fishtail bit is used, having above it five reamer blades which cut a 30-in. hole. This is shown in Fig. 1. The bit is fitted to a drill rod rotated by the well-drilling machine. The cuttings are washed to the surface by a hydraulic jet. If hard rock is encountered, the drill bit is replaced by a core barrel 6 to 20 ft. long. Steel shot is fed into the wash water and carried under the edge of the barrel, which thus grinds and cuts an annular hole. The core is broken off at intervals.

A well-drilling outfit built by the American Well Works for use on this irrigation work in Texas is shown in Fig. 2. The derrick is 45 ft. high, and within it the hollow drill rod is suspended by the bail shown in Fig. 1.

The rod is gripped by a horizontal rotating device at the head of the well, this being driven from the engine by a sprocket chain and making 40 to 100 r.p.m., according to the character of the material encountered. The drill rods are lowered by a friction brake, the feed depending on the rate of drilling and the removal of the material by the water. Lengths of rod are added as required.

The wash water is delivered through the drill rod, passing out through two small holes in the fishtail bit (Fig. 1). In order to economize in the use of this water (the supply being usually limited), the discharge is delivered to a long ditch, or flume, in which the cuttings are deposited. The clear water flows into a basin, from which it is returned to the pump.

Working in the formation described above, with no hard rock, this plant has sunk wells 250 ft. deep in an average of 3½ days' actual drilling. With the time consumed in erecting and dismantling the derrick and machinery, this gives about a week for each completed well.

Underflow Water-Supply at Moline, Kan.

By W. L. Benham*

The city of Moline, Kan., with a population of about a thousand, has recently developed a water-supply from a shallow underflow source by means of two 200-ft. lengths of collecting pipe and a receiving well 24 ft. in diameter and 32 ft. deep. The water thus secured is pumped to a tank and distributed thence through the city.

The city lies in the valley of Wild Cat Creek, which goes dry during extended droughts. Surveys made in the early part of 1913 proved that it was possible to impound water for storage, but owing to the excessive cost of dams and right-of-way and to the fact that the State Board of Health requires the filtration of surface supplies such a proposition was beyond the means of the city.

The logs of gas wells which had been drilled in this vicinity showed the presence of salt water at a depth of 80 ft., and no better water between that depth and 1200 ft., which put deep wells out of the question. There were at that time three wells, each about 10 ft. in diameter, at points in the city forming a triangle approximately 3000 ft. to the side and from these wells and from rain-water cisterns was obtained the entire water-supply of the city. Levels connecting these wells showed that the elevation of the water-bearing stratum in each was approximately the same, and as they were all on the north side of a small branch of Wild Cat Creek, it was decided to investigate the possibility of securing an adequate supply for the city from the stratum that fed these wells.

With this object in view, separate pumping tests were made of the wells, making use of small boiler-feed pumps and a 2-in. steam jet, the steam for such tests being furnished by a 20-hp. steam traction engine. The result of 10-day tests led to the selection of some point to be located in the vicinity of a well on the south side of the city.

By means of careful borings across the valley it was determined that the flat was underlaid at a depth averaging 11 ft. with a layer of hard blue limestone, upon which lay from 7 to 9 in. of coarse gravel, which in turn was overlaid with black gumbo, very hard and practically impervious to water. Under the limestone, which was about 4 ft. thick, was a layer of black, slaty shale which was known to be not less than 60 ft. deep and absolutely dry. The borings demonstrated that the water-bearing gravel stratum was approximately 400 ft. in width, parallel to the branch of the creek, but that a fault in the rock cut the underflow off from the main stream.

From the data thus secured, it was decided to build an impounding well with a capacity of approximately 200,000 gal. and to intercept the flow and conduct it to the well in vitrified clay pipe, laid with open joints.

In July, 1913, work was started at the lowest point of the underlying rock ledge on the excavation for the well, the city doing the work with hired teams and labor. Excavation was done with teams to a depth of 7 ft., when the soil became too wet and sticky for such methods. The balance of the excavated material was handled with a derrick having a 35-ft. mast and a 40-ft. boom, the mast being swung by hand and the material being raised in one-half-yard skips handled with a single-drum steam hoist.

When excavation reached the rock layer, it became necessary to install a steam pump, which was run continuously. The rock was drilled with steam tripod drills and shot with small charges of 60% dynamite, all stone being broken into small pieces and great care being taken to prevent shattering the banks. Excavation above the rock was circular in shape and 34 ft. in width at the bottom. Excavation in and below the rock was trimmed to a net diameter of 25⅓ ft. The excavation in the shale was easily handled, it being deemed economical to loosen it with small shots of 30% dynamite instead of by picking.

At a depth of 16 ft. below the rock line, excavation was stopped and a 6-in. concrete bottom, draining to the center, was laid. The balance of the excavation in the shale was then lined with 8 in. of hard burned brick laid in cement mortar, this being done because the shale softened under the action of water.

After lining to the rock level was completed, work was commenced on the two galleries. These were each 200 ft. in length, laid north and south at right angles to the underflow and entirely in the rock ledge below the water-bearing gravel. These were 8-in. salt-glazed, vitrified clay sewer pipe, laid to a 0.5% grade, the joints being loosely packed with untarred rope yarn and the pipe being laid on a 6-in. bed of broken stone. The pipes were surrounded and covered to a depth of 1 ft. with broken stone and the downstream wall of the ditch was backfilled with stiff clay tamped into place, the object being to stop all flow across the galleries. The galleries were laid true to line and grade, and an observation manhole was placed at the end of each line.

While work was being done on the galleries, the lower portion of the well was cleaned out and the wall run up from the rock level to a point 6 ft. above the surface of the ground. This portion of the well was 30 ft. inside diameter and was walled with 15 in. of brick, plastered on the outside.

As the city was at the same time constructing an electric-light system, it was decided to install a 250-gal.-per-min. motor-driven extended-type triplex pump and a 150-gal.-per-min. pump of the same type for emergency use, to be driven by a gasoline engine. These were supported on I-beams over the wall and the well and the pumphouse above it were covered with a timber and shingle roof.

Observations for a few hours indicate that the galleries yield water at a rate of about 110 gal. per min., which if

*Chief Engineer, Benham Engineering Co., Oklahoma City, Okla.

continuous would give a daily capacity of more than 150,000 gal.—far more than is necessary for a city of a thousand inhabitants. The quality of the water is excellent for domestic purposes, and the water and the method of securing it have been approved by the Kansas State Board of Health.

The work of investigation, the preparation of designs and the superintendence were done by the Benham Engineering Co., Consulting Engineers, Oklahoma City, Okla.

The Cabrillo Bridge at the San Diego Exposition

A concrete bridge, architecturally a series of severely plain arches, forms one of the principal approaches to the Panama-California Exposition, now being held at San Diego, Calif. In fact this viaduct, known as the Cabrillo Bridge, is not a series of arches but rather a series of hollow, box-like pedestals with the upper parts cantilevering out to form the arched openings. As faintly shown in the view in Fig. 1, there is a joint at each arch-crown, indicating the cantilever construction. The bridge is 946 ft. long, and its main portion comprises seven semicircular 56-ft. arches, with a maximum height of about 120 ft. to the roadway.

The purpose of this peculiar design is explained by Frank P. Allen, Jr., Director of Works. The architectural style of the exposition is the Spanish-Colonial, and in order that the bridge might be in keeping with this it was necessary to follow the general design of the old Spanish bridges. With few exceptions (where pointed Gothic arches were used) all Spanish masonry bridges until modern times have been built with full semicircular arches of comparatively short spans. Semicircular arches carried on tall and relatively slender piers will generate in the piers bending moments that will

FIG. 1. THE CABRILLO BRIDGE AT THE SAN DIEGO EXPOSITION
(A series of reinforced-concrete cantilevers in the form of arches)

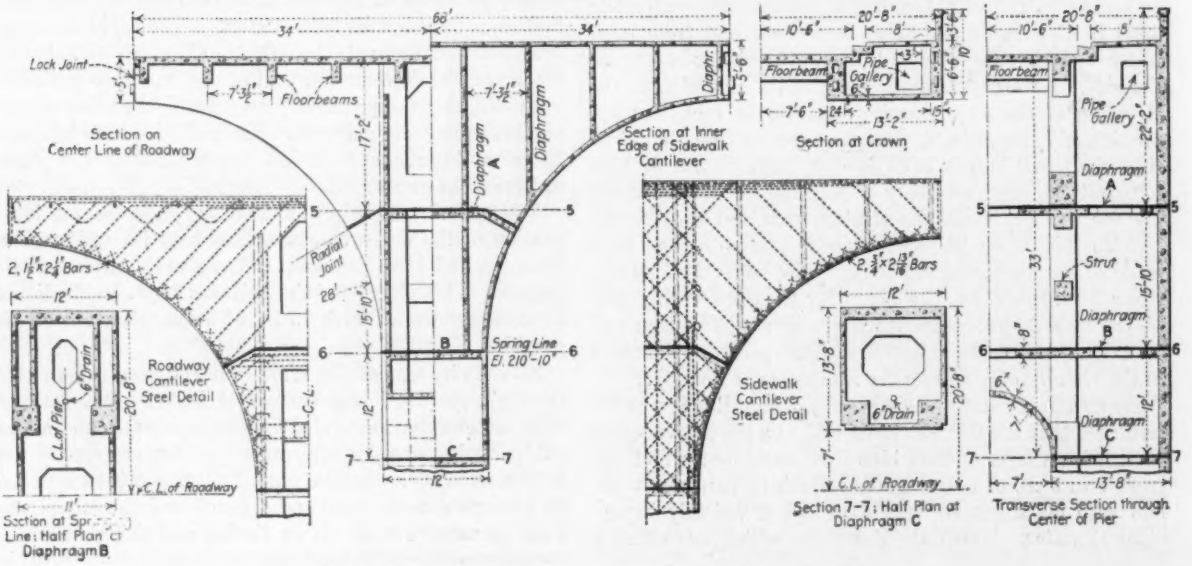


FIG. 2. DETAILS OF SUPERSTRUCTURE, CABRILLO BRIDGE

be expensive to deal with. When temperature stresses are added to these moments, for a bridge of this length the cost of the structure becomes almost prohibitive.

For this reason it was decided to use the spandrel

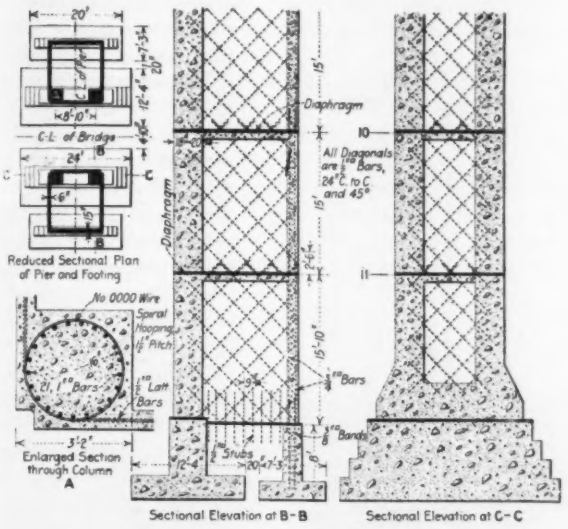

Reduced Sectional Plan of Pier and Footing
Diaphragm
All Diagonals are 1" Bars, 24" C. to C. and 45°
No 0000 Wire Spiral Hooping 4" Pitch
Cl. 1" Bars
1" Lat. Bars
1" Bars
Enlarged Section through Column A
Sectional Elevation at B-B
Sectional Elevation at C-C

FIG. 3. DETAILS OF PIERS, CABRILLO BRIDGE

walls as cantilevers, cutting the bridge completely through at the crown of each arch and making each section (a pier and two half arches) entirely independent of the others. For the outer cantilevers (carrying the sidewalks only) the resultants fall well inside the middle third of the pier, and so are carried on a wall designed as an ordinary pier. For the roadway cantilevers the resultants fall considerably outside the middle third of the width of the pier. On this account Mr. Allen substituted two reinforced-concrete columns for the solid pier, and designed the cantilever as over two supports.

The bridge is designed to carry a uniform load of 100 lb. per sq.ft. and two moving loads of 15 tons each, passing each other at any point. The foundations are simple and are designed for a soil loading of 3 tons per sq.ft. They rest on a bed of sea sand which is compact enough to require picking for removal. Water was encountered at the three center piers only, and was easily handled with coffer-dams and small pumps. The greatest depth of excavation was 23 ft.

The detailed construction is shown in Fig. 2. There are four cantilevers rising from each side of each pier, two, in pairs, being connected by transverse vertical diaphragms and by a slab forming the soffit of the arch. The arch ribs thus formed are connected by floorbeams supporting the roadway slab, so that in effect the whole overhang is a box. The depth at crown of arch is 5 ft. for the inner or roadway cantilevers and 6 ft. 6 in. for the outer or sidewalk cantilevers. The anchor arms

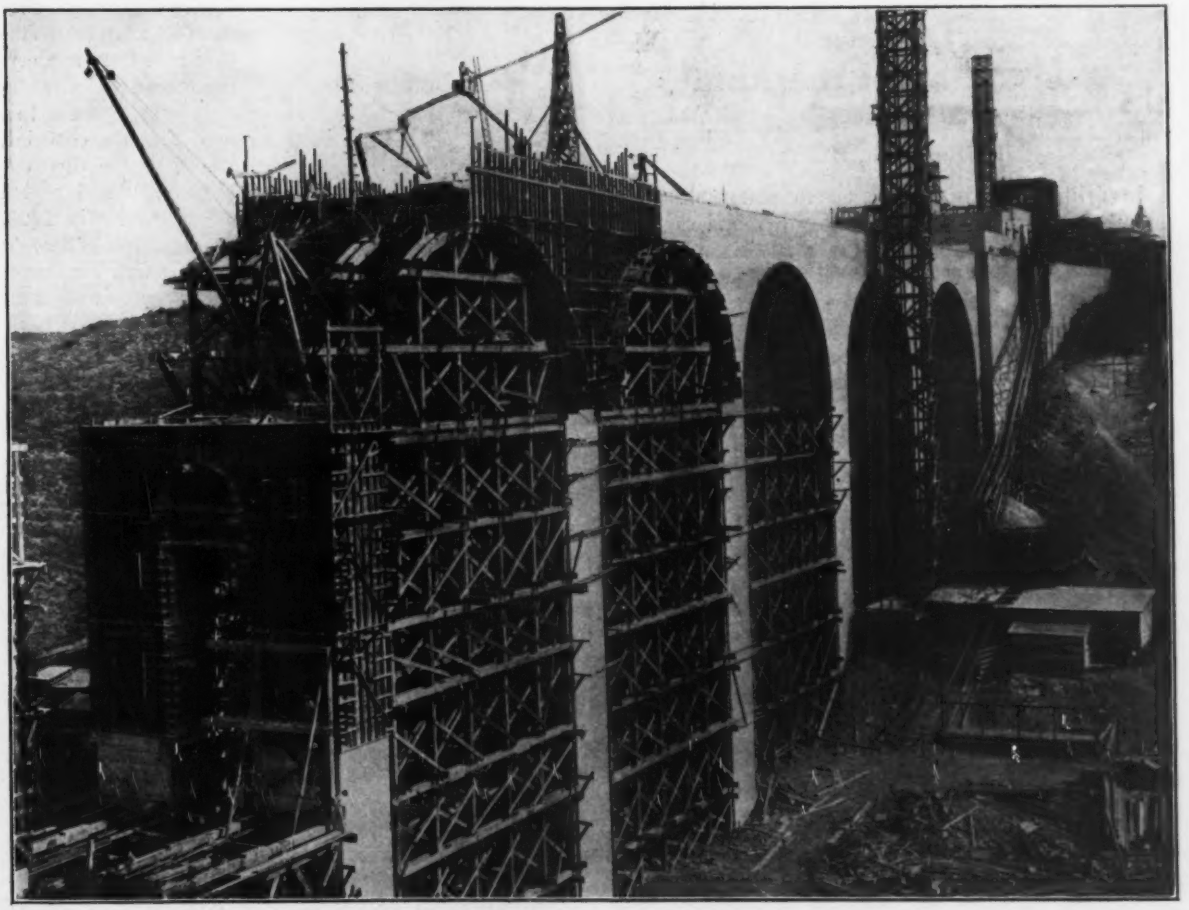


FIG. 4. THE CABRILLO BRIDGE AT THE SAN DIEGO EXPOSITION UNDER CONSTRUCTION

of the end piers are triangular in form, springing from the level of diaphragm 5 (Fig. 2). These are concealed by the curtain walls of the abutment.

Each pier consists of a pair of hollow columns, with a width the same as that of the arch ribs, and the cantilever arms spring from the sides of these columns. The cross-section is proportioned to the distribution of loads, the inner sides (carrying the roadway cantilevers) being much heavier than the outer sides, which carry the sidewalk cantilevers. Above the springing line of the arches the columns are connected by curtain walls which extend between the arch ribs to the level of the roadway. The construction of the piers and footings is shown in Fig. 3.

The approaches have girder spans on columns, concealed by curtain walls which give the appearance of ordinary abutments.

The bridge was designed in the engineering department of the exposition, under the direction of Mr. Allen, and was built by the exposition forces. The total cost of the bridge and approaches was $195,653, including the overhead charges for design and supervision.

The materials were rather high in price. Crushed stone and sand had to be hauled partly by rail and partly by automobile truck; the haul was 15 and 22 miles respectively, and the cost delivered was $2.06 per cu.yd. for the stone and $1.625 for the sand. Cement cost $2.12 per bbl., and lumber averaged $20 per 1000 ft. b.m., both delivered at the work. Carpenters and laborers were paid respectively $4 and $2.25 per 8-hr. day.

The Cone Bit for Rotary Drilling*

By Howard R. Hughes†

In drilling for water and oil to reasonable depths through the generally soft yielding clay and sand formation of the coastal plain of Texas, Louisiana and Mississippi, the rotating method of drilling was adopted, principally on account of the easy and quick penetration and the low cost of the drilling plant.

In favorable ground, free of heavy gravel and rock strata, as much as 1000 ft. has been drilled in less than 36 hr., although such performances were of course rare.

Hydraulic rotary drilling, or as it is now called rotary drilling, was used in the above states as early as 1880. The plant consisted of an ordinary derrick, a 25-hp. boiler, a small hoist, a steam pump and a water swivel with hose attachment and an ordinary flat diamond-pointed bit.

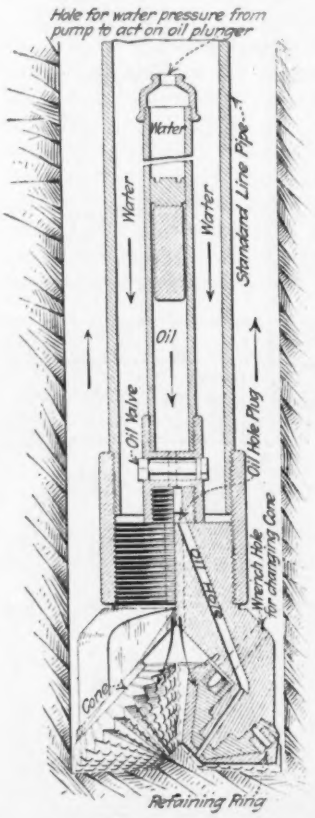
The method, as its name implies, involves the rotation of a pipe by means of machinery placed on the derrick floor. A drill bit attached to the lower end cuts a clearance for the drill pipe, with much the same motion and effect as an auger. Water forced through the drill pipe by means of a pump and escaping through the bit removes the cuttings and returns to the surface outside the drill pipe. In this manner the hole is kept open, permitting the drill stem to rotate freely. The pressure of the column of muddy water holds up the walls of the hole until it has been cased.

*Presented at the annual meeting of the American Institute of Mining Engineers in New York City, February, 1915.

†Houston, Tex.

Practically all the wells of the gulf-coast region, numbering nearly 10,000, have been drilled with this system. During the last five years its use has been extended to many other states and countries.

The great drawback hitherto has been the slow progress made in drilling rock and other hard formations. For soft, caving formations no other system can approach it in efficiency.



Sharp & Hughes Cone Bit in Hole

The old-style bit in general use is known as the fish-tail type. It has only two cutting edges and soon grinds down flat when hard rock is encountered. At times only a few inches a day can be made with it. The racking and wrenching to which the machinery and drill pipe are subjected, when drilling a hole from 4 to 18 in. in diameter, result often in "twist-offs" of the drill stem and costly fishing jobs. With the use of the heavier rigs and in deeper drilling the need for a bit adapted to cutting rock became a crying necessity.

It was to meet this need that the cone bit, known as the Sharp & Hughes, was invented by the writer in 1908. This consists of two or more detachable, cone-shaped cutters of hardened steel. These cutting cones revolve on bronzed bearings, lubricated with a special heavy viscous oil supplied by means of a small pipe carried inside the drill stem. The cutters, being detachable, may be removed and sharpened when dull.

The principle of the cone bit is entirely different. The edges, or cone points, of the bit roll in a true circle like a cone bearing, and crumble or chip away the rock. The cone points, being of very hard steel, wear away slowly. Often they show only slight wear after drilling 50 ft. of rock.

The proper adjustment of the weight upon the bit is the secret of good work with this drill. Experience has shown that the following weights give satisfactory results for the corresponding bits of standard sizes:

DRILLING WEIGHT ON BIT

Diam. of Bit, In.	Weight, Lb.	Diam. of Bit, In.	Weight, Lb.
2⅜	2,870	6½	7,900
2½	3,480	7⅛	8,650
3⅞	4,700	7⅝	9,270
4⅜	5,310	7⅞	9,560
4⅝	5,910	8½	10,300
5½	6,670	9	11,000
5⅞	7,140	9⅞	12,000

For bits larger than those in the table, as much weight as practicable may be employed with little or no risk of

overloading the bearings in the bit; but within the limits of the table, the weights given are probably as great as prudence would permit. For extremely hard rock, the speed, not the weight, should be increased.

While primarily designed for oil and water wells, this cone bit can be applied in drilling sump holes for mine pumps, in making air shafts and in driving holes inclined at various angles from the vertical. Its use greatly enlarges the field of the rotary system (and the cone bit already is extensively used in California, Mexico, Trinidad, Roumania, Russia, Persia, Egypt, Japan, Borneo and India) by presenting the great advantages of more rapid drilling through hard rock, of reaching greater depths than any other rotary apparatus can compass, of finishing a hole with smaller reduction of surface diameter than any other system permits, of the consequent requirement of fewer "strings" of casing, of less deterioration of drill pipe through stresses and vibration, and of the saving of much time consumed in removing the drill pipe for sharpening or changing bits.

A comparison of rotary and standard drilling costs as deduced from recent tests in California is given on p. 943 of this issue.

A Small Bascule Bridge; Pere Marquette R.R.

A small trunnion bascule bridge for a span of 46 ft. 3 in. between piers and a clear waterway of 43 ft. 3 in. is an interesting structure on the Pere Marquette R.R. at Benton Harbor, Mich. The bridge crosses the Paw Paw River, and has a 50-ft. fixed span (with 76-in. girders) in addition to the bascule span. Both are double-track deck spans. The bascule is peculiar in consisting of two separate parallel single-track spans which may be operated together or independently.

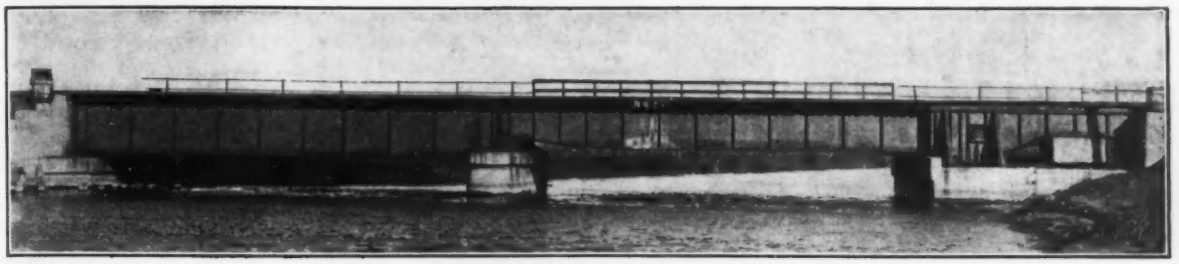


FIG. 1. 50-FT. TRUNNION BASCULE BRIDGE OVER THE PAW PAW RIVER AT BENTON HARBOR, MICH.; PERE MARQUETTE R.R.

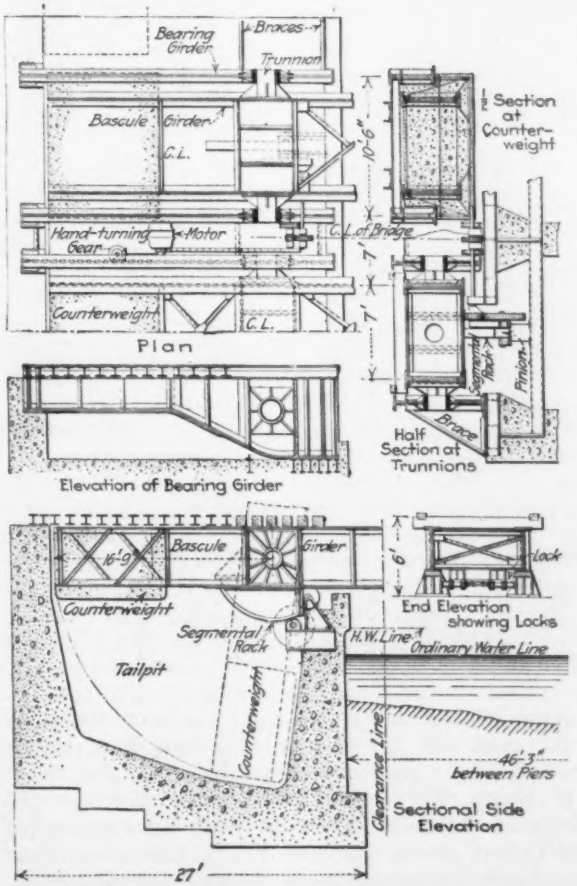


FIG. 2. DETAILS OF TRUNNION BASCULE BRIDGE; PERE MARQUETTE R.R.

The bridge is of simple appearance, with all mechanism, etc., housed below the deck, as is shown in Fig. 1. The general design and the arrangement of the operating machinery are shown in Fig. 2. The bridge is proportioned for Cooper's E-60 loading. It was designed and built by the Wisconsin Bridge & Iron Co., of Milwaukee.

The bridge is of the fixed-trunnion type, with trunnions carried in bearing girders which span the tailpits. The bascule girders are 70 ft. 9 in. long, with a main arm of 54 ft. and a counterweight arm of 16 ft. 9 in. They are 54 in. deep, reduced to 39 in. at the end of the leaf. The tail ends are connected by cross-frames and carry a counterweight block of concrete having steel punchings mixed with it to give a weight of about 300 lb. per cu.ft. On this end the deck has steel ties, with an 18-in. plate and two 3x3-in. guard angles for each rail. The main arm has wood ties and guard timbers.

For operating, there is a 25-hp. motor, with chain drive to two gear trains, each ending in a pinion which engages a segmental rack beneath the deck and in the center-line of the span. Hand gear is provided also. The bridge can be opened as a single unit by power in about 30 sec. Hand operation requires 12 min. for each leaf. The end locks are bolts moving laterally and entering slots in the shoes of the bridge seats, as shown in the end elevation.

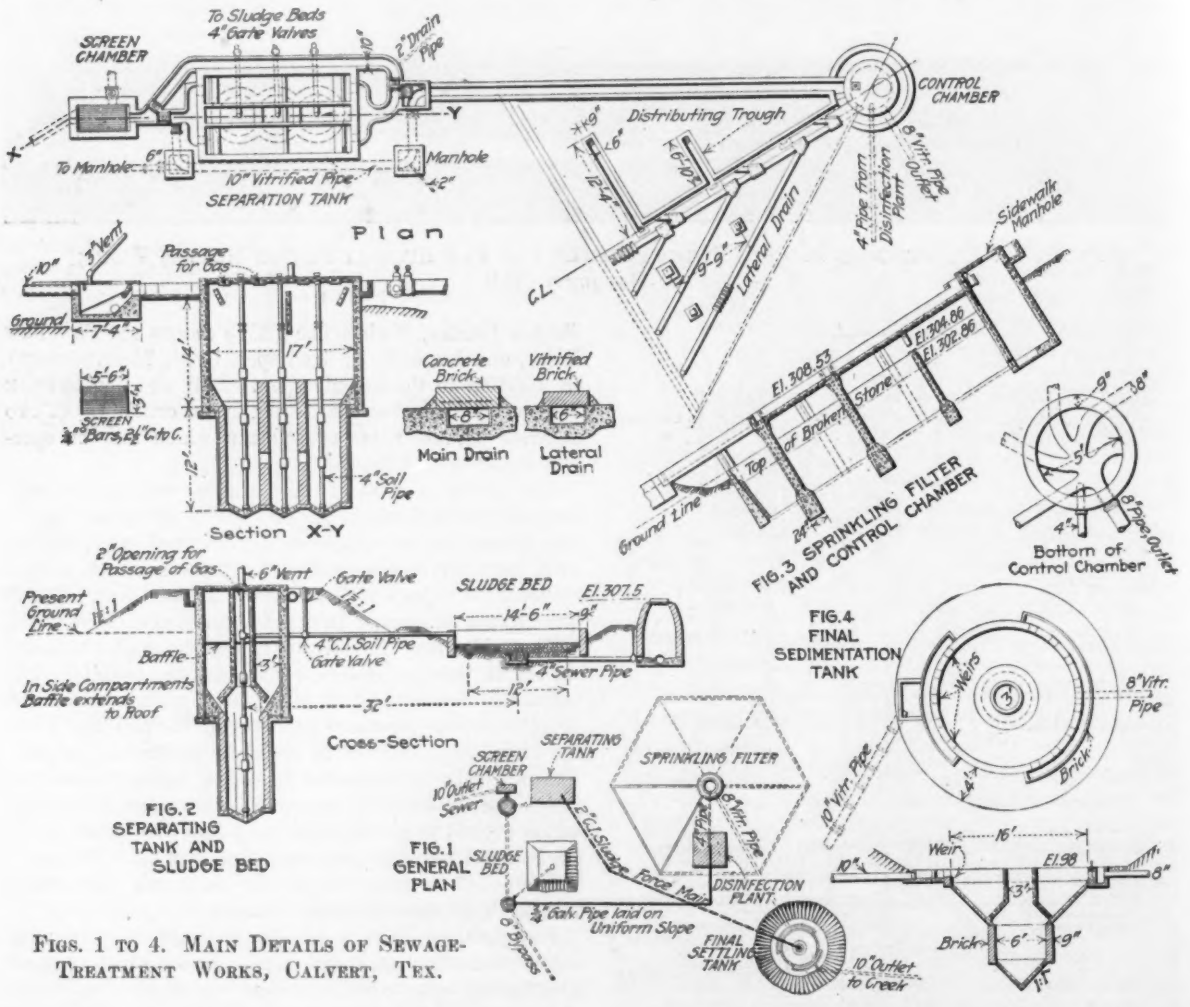
The piers are of concrete. The pier carrying the bascule span has a tailpit for each leaf; here the concrete was mixed with a waterproofing compound so as to prevent seepage of water into the pits.

Sewage-Treatment Plant at Calvert, Tex.

By Thomas L. Fountain*

SYNOPSIS—An unusually complete and carefully detailed plant. It consists of bar screens, a two-story settling tank, double secondary screens, sprinkling filter, disinfection house and final settling tank.

The sewage-treatment works of Calvert, Tex., is a small plant designed by the writer to care for an exceedingly irregular flow of fresh domestic sewage. The plant serves the 1500 white inhabitants of the town. Its total cost, exclusive of the site, was nearly $5000.



FIGS. 1 TO 4. MAIN DETAILS OF SEWAGE-TREATMENT WORKS, CALVERT, TEX.

The Sewerage System

To avoid pumping and to make possible any treatment besides sedimentation many of the sewers were laid on a 0.2% grade and the 10-in. outlet on a 0.15% grade. All manholes in the system have tight covers. Ventilation of the sewers is secured through the house connections, which are left untrapped. Flush tanks located at the upper ends of all sewer lines are operated once each week by an attendant. The six miles of 10- to 6-in. sewers have been in use for more than a year without being cleaned or clogged.

Conditions Governing Design

Tidwell Creek, the small watercourse which receives the effluent from the treating plant, has a dry bed except during time of rainfall and for a few hours thereafter. The creek bed has practically no fall, either in the vicinity of the plant or for several thousand feet below. It consists of a series of wide shallow holes connected by small channels. One of the main highways leading to Calvert crosses this creek 700 ft. downstream from the plant. The site selected for the sewage-works is 700 ft. from the nearest house and the same distance from the public road. The problem then became one of securing

*The Fountain-Shaw Engineering Co., College Station, Tex.

without assistance from dilution a fairly clear effluent of uniform stability through the treatment of the sewage in a plant which, while simple and reliable in operation, could be depended upon not to create a nuisance.

The available head of only 10.8 ft. indicated separation tanks and double-contact beds for the main treatment, but studies showed a sprinkling filter to be much more desirable, on account of its lower first cost and freedom from the many troubles of operation peculiar to double-contact beds. It was found that by saving every possible inch of head a sprinkling filter could be used, and it was accordingly adopted.

To avoid the very objectionable odors which the writer has observed at uncovered screen chambers and separation tanks elsewhere in Texas, those at Calvert have been tightly covered, the gases of decomposition being liberated through a 60-ft. vent pipe. These covers also shut off from the sight of visitors the disagreeable mass collected on the screens and the scum formed on the sewage in the separation tank.

Since all houses to be served by the sewerage system were already supplied with water and meters were being

FIGS. 5 TO 7. VIEWS OF SEWAGE-WORKS, CALVERT, TEX.

(5, General view, showing primary or bar-screen chamber in foreground, separation or two-story settling tank, ventilation pipes from the two tanks, sprinkling filter, and disinfection house. 6, Distributing troughs and spattering desks of sprinkling filter. 7, Final settling tank, with channels and weirs.)

installed throughout the town, the total daily sewage flow was estimated from the pumping records to be not more than 25,000 gal. As practically all of the sewage would be supplied from residences, a very irregular rate of flow was anticipated. The average rate of flow was taken as 37,500 gal. per 24 hr. All of the sewers are above ground-water level, so no allowance was made for infiltration. Since the population of Calvert has shown no increase for a number of years, no special provision for additions to the treatment plant was made where such provisions would increase the first cost.

GENERAL DESCRIPTION OF SEWAGE-WORKS—The complete plant now constructed and in operation consists of a screen chamber, a separation tank, a sprinkling filter, a final sedimentation basin, a sludge bed and a disinfecting plant. The entire plant is surrounded with a small levee to prevent inundation from Tidwell Creek in time of flood. The general plan of the works is shown in Fig. 1 and a general view in Fig. 5.

PRIMARY SCREEN CHAMBER

The primary screen chamber (see Fig. 2) is equipped with removable inclined bar screens, in two sections. This chamber is fitted with a gas-tight circular hinged steel cover, held tightly against composition rope packing by four wedge-buttons, cam-shaped in plan, provided with lever handles for rotating in a horizontal plane.* Gases from this chamber are carried to the 6-in. main vent stack through a 3-in. standard galvanized-iron pipe. The outlet end of the chamber is inclined with a view of causing decomposable matter settling upon it to slide toward the center of the chamber so that gases of decomposition will pass off through the 3-in. vent pipe. The inclined concrete slab near the outlet end, extending from the top of the chamber to a point 6 in. below the normal sewage level, serves as a trap to prevent the passage of gases from the screen chamber to the air at the ground level.

SEPARATION TANK

A separation tank (Fig. 2) with a flowing-through period of 2 hr. at the rate of 37,500 gal. per 24 hr., the hourly flow $^1/_{18}$ the average flow, or 2100 gal., would have been sufficient for present needs, but as twice this capacity could be secured at only slightly increased cost, and the construction difficulties in the smaller tank would have been greater, the tank as finally designed and built is of sufficient size to care for a sewage flow of 75,000 gal. per 24 hr. The inlet and discharge openings of one of the two settling compartments are sealed for the present. The entire decomposition chamber below the slots in the division walls has a capacity of approximately 560 cu.ft., permitting sludge storage for a period of three months from a sewage flow of 75,000 gal. per day, each million gallons of this dilute domestic sewage being estimated to furnish 80 cu.ft. of wet sludge. For the present daily flow of 25,000 gal., the sludge from both the incoming sewage and that from the sprinkling-filter effluent, estimated as 135 cu.ft. per million gallons, can be stored for five months.

When the excavation for the construction of the tank had reached a depth of about 10 ft. below the original ground surface, fine, running dry sand was encountered. The brickwork for the three sludge wells was then carried to its full height and the whole surrounded with a reinforced-concrete shell. The sand was then removed from each well simultaneously, the entire brick and concrete mass following the excavation by its own weight.

The upper edge of each inlet and outlet opening of the separation tank extends 8 in. below the normal sewage level in the tank, to prevent the escape of gases. Openings at the top of the division walls between the decom-

*An illustrated note on these covers was printed on p. 735 of our Field and Office section for Apr. 15, 1915.

position chamber and the settling chambers permit all gases to pass to the 60-ft. galvanized-iron vent stack erected over the center of the tank.

Reversal of flow through the settling chamber and by-passing of settled sewage are accomplished by channels and gate valves.

Six manholes give access to the tank for removing scum and for cleaning. The steel-plate covers are drawn tightly against the rope packing with bolts set in the concrete roof of the tank. As a further precaution against the escape of gases, the packing is thoroughly covered with pitch.

The sludge is conveyed from the bottom of each well by a 4-in. cast-iron soil pipe controlled by a gate valve. Each pipe extends to the top of the concrete cover. The upper end is closed with a standard plumber's cleanout plug, which can be quickly removed and replaced without the use of special tools.

Sludge Bed—The sludge bed (Fig. 2) has a net area of 144 sq.ft. The bed should receive one filling per month of 100 cu.ft. from the present estimated daily flow of 25,000 gal.

Sprinkling Filter

One triangular sprinkling-filter unit (Figs. 3 and 6), capable of caring for a sewage flow at the rate of 37,500 gal. per 24 hr. on a 2,000,000-gal. per acre per 24-hr. basis, has been constructed, out of a possible six forming a hexagon. The average depth of broken stone is only 5 ft., one of the compromises made necessary by the extremely limited total available head for the plant. The side walls are carried up 2 ft. above the surface of the bed to retain the spray.

Secondary Screens—To remove floating matter from the tank effluent before it goes onto the sprinkling filter, two small vertical screens are provided at the central chamber. The first of these is of galvanized-iron wire, 4 meshes per inch, and the second of 14-mesh bronze wire. These screens are removed and cleaned daily by the use of a scrubbing brush and a bucket of sewage.

Control Chamber—The reinforced-concrete control chamber (see part of Fig. 3) will serve the entire filter area, should occasion arise for the construction of the additional units. Settled sewage is conveyed through a reinforced-concrete channel to the circular channel at the top of the control chamber, from which it flows through a similar channel to and through vertical discharge tubes on to spattering disks and thence is sprayed over the filter. After passing through the broken stone the effluent is collected and returned through the underdrains to the bottom of the central well.

Discharge Tubes—The discharge tube shown in Fig. 8 is the result of experimentation with discharge tubes in several plants designed by the writer. A tube having an interior diameter of ¾ in. was found unsatisfactory on account of excessive discharge when the depth of flow in the distributing channel was sufficient to give anything like uniform distribution to the tubes, and also on account of extreme irregularity of discharge during times of small flow, due to the inability of the sewage to fill the tube bore. When ½-in. tubes were substituted, these defects were practically eliminated. Experience with the smaller tube proved that clogging is no more frequent than with the larger size. Trouble had also been experienced from the lodging of floating patches of soft scum against the sides of the tubes, particularly at times of low flow, which frequently stopped their operation. In other plants no provision had been made to prevent the action of the wind on the column of sewage falling from the tube to the disk, with the result that high winds kept the sewage off the disk a large part of the time. In the design of the Calvert discharge tube, these defects in the former tubes were eliminated as far as possible.

The Calvert discharge tube has an interior diameter of ½ in. and a total length of 12 in., the upper 9 in. of which is threaded. The upper end terminates in a brass orifice plug so shaped as to give a conical approach to the tube to facilitate the entrance of bits of scum and small objects suspended in the sewage. The tube is supported by being screwed through a brass plug, which is screwed into a brass coupling embedded in the concrete bottom of the distributing channel.

The tube can be screwed up or down through its supporting plug by use of a stillson wrench. The top of the orifice plug is never raised above the bottom of the distributing channel. The wind guard, composed of a 4-in. pipe, extending to within 6 in. of the spattering disk, is bolted to the bottom of the concrete channel.

Spattering Disks—The type of disk used for distributing the sewage over the broken stone at Calvert was devised by C.-E. A. Winslow and Earle B. Phelps.*

The estimated normal discharge upon each disk for the Calvert plant is 4.08 gal. per min., the total head nearly 5 ft., the head on the sprinkler approximately 2.8 ft., and the distance between disks 11 ft. 4 in., which conditions should give a "corrected coefficient"† of about 0.5, a very satisfactory value considering the simplicity and accessibility of this scheme of gravity distribution.

To operate successfully, the spattering disk must be supported rigidly and independently of the filter material. It must also be arranged for sidewise adjustment, with provision for fastening after its proper position has been determined. The form of construction shown in Fig. 8 meets these requirements satisfactorily. The disk, turned from solid brass, rests on a cast-iron base plate and is controlled by three brass screws. The base plate is supported by a 3-in. galvanized-iron pipe set in an 8x8-in. concrete pier.

Underdrains—The underdrains (Fig. 3) are constructed in the filter floor. The main drain has a grade of 0.70%, while that in the laterals is slightly less. Contrary to the usual practice in this country, the laterals are not closely spaced, the distance between center lines being 5 ft. 4 in. This wide spacing produces a considerable depth and velocity of flow in each lateral drain which insures the removal of suspended matter that would otherwise collect on the filter floor. A slight current of air is also created above each drain, which is probably of some assistance in expelling the carbonic acid gas formed in the filter. Aëration of the bed is accomplished by having the drains much larger than necessary for carrying off the water, the upper half of each drain acting as an air duct. The upper end of each drain is connected with the outside air through a 3-in. cast-iron soil pipe extending 12 in. above the top of the broken stone.

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*"Studies of Sewage Distributors for Trickling Filters," Contributions from the Sanitary Research Laboratory of the Massachusetts Institute of Technology, Vol. IV, 1908.

†The corrected coefficient is obtained by multiplying the coefficient of distribution by the ratio of the circular wetted area to that of its circumscribing square (see "A Method for Testing and Comparing Sewage Sprinklers," by Earle B. Phelps, "Engineering News," Vol. 56, Oct. 18, 1906).

Filter Material—The bottom 9 in. of filter material consists of hard, overburned, broken brick. The largest pieces are not over 4 in. in greatest dimension and the smallest not less than 2 in. in least dimension. The remainder of the filter material is hard, tough, broken limestone, known as 1¼-in. stone. It ranges from 1¼ to 1½ in. in greatest diameter. A clearer effluent could have been obtained by using ½-in. stone for the upper 12 to 18 in. of the filter bed, but serious trouble from clogging of the bed would very probably have resulted. Great

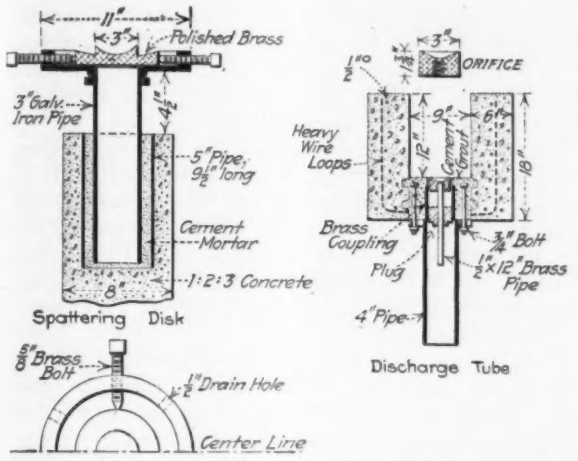


Fig. 8. Sprinkler Details, Calvert Sewage-Works

care was taken to prevent any dirt or stone dust from getting into the filter with the broken stone.

Final Sedimentation Basin

The final sedimentation basin (Figs. 4 and 7) is circular in plan, this shape having been adopted to facilitate the collection of the sludge precipitated from the oxidized sewage after its passage through the sprinkling filter. The sedimentation chamber is separated from the sludge well by a reinforced-concrete funnel-shaped dividing wall, the lower edge of which deflects gases from the sludge well and prevents them from interfering with sedimentation. The sedimentation chamber is of sufficient capacity to care for a sewage flow at the rate of 37,500 gal. in 24 hr., with a flowing-through time of two hours. The sludge well below the lower edge of the funnel-shaped dividing wall has a capacity of 150 cu.ft., permitting the retention of sludge for three months from a daily sewage flow of 25,000 gal., the rate of accumulation being estimated as 54 cu.ft. per million gallons of oxidized sprinkling-filter effluent.

A hand pump mounted on the top of the dividing wall is used to raise the sludge from the bottom of the well and force it through a 2-in. galvanized-iron pipe into the decomposition chamber of the separation tank.

Weirs—The sprinkling filter effluent is admitted to the final sedimentation tank, and the effluent from this tank is discharged over a number of weirs, each consisting of a cast-iron plate having an approximately true upper edge. This plate is held in position by two bolts set in the concrete wall of the basin. The bolts pass through slotted openings in the plate, and the weir crest can be accurately adjusted at any time by loosening the nuts on these bolts.

Disinfection Plant

This plant is quite similar to the one constructed for the disinfection of the sprinkling-filter effluent of Pleasantville, N. Y., which is particularly adapted to the service required on account of its simplicity and reliability of operation under unskilled and infrequent attention. Hypochlorite of lime (commercial bleaching powder) is the disinfecting agent employed.

Each duplicate unit for mixing and storing the hypochlorite has a capacity of 75 gal. of solution.*

Mixing and Storage Tanks—Each mixing tank (Fig. 9) consists of a 2½-ft. length of double-strength sewer pipe, with a 6-in. Y-branch for an outlet. The diameter of the outlet is reduced to 1½ in. A wooden plug is used to close the opening while the solution is being mixed, and a wooden trough to convey the solution to the storage reservoir. The mixing is done with a wooden paddle. Pure water for the solution is hauled from the

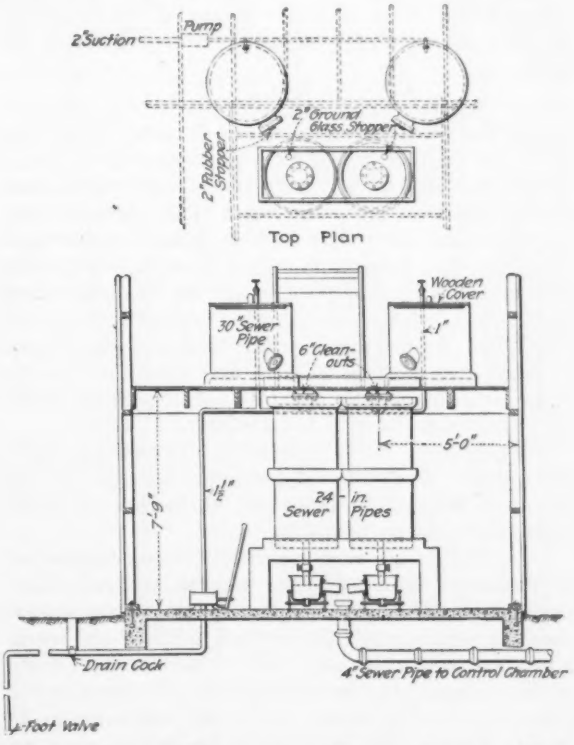


Fig. 9. Sewage-Disinfecting Plant, Calvert, Tex.

city water-works and stored in a concrete underground cistern holding 800 gal., which is located just back of the building. From this cistern the mixing tanks are filled with a hand pump.

Each storage tank is made from two 2½-ft. lengths of 24-in. double-strength sewer pipe, carefully jointed and closed bottom and top with 1 to 1 cement mortar. These tanks rest on a reinforced-concrete table, under which are located the orifice tanks. A 2-in. opening for receiving the

*As each charge is to last 24 hr., the solution must be discharged from the orifice tank at the rate of 0.052 gal. per min. Assuming that the bleaching powder contains 33% of available chlorine and that 4 parts of available chlorine per million are required for satisfactory bacterial removal, 100 lb. of the bleach is required per million gallons of sprinkling filter effluent ("Sewage Disposal," Fuller). On this basis the Calvert works, when handling a sewage flow at the rate of 37,500 gal. per 24 hr., would require 3.75 lb. of bleaching powder each day.

solution from the mixing tank was provided by embedding a ground-glass bottle neck in the mortar top. This opening is closed with a ground-glass stopper when the storage tank is in service.

ORIFICE TANKS—The automatic-feeding device for the application of the solution to the sprinkling filter effluent is a cast-iron slop sink, 14 in. sq. and 11 in. deep, enameled on the inside, and provided with a ⅛-in. orifice drilled in one side 4½ in. from the top. This tank rests on a cast-iron plate 18 in. square, which is supported by three screw legs resting loosely in small iron caps set in the concrete floor of the building. The plate, together with the orifice tank, can be raised or lowered by these screws until the head on the orifice produces the desired discharge. A short piece of 3-in. iron pipe, fitted with a flange for bolting to the side of the orifice tank, conducts the solution to the mouth of a 4-in. vitrified pipe, leading to the control chamber, in the bottom of which the solution is added to the filter effluent.* A cement-mortar channel in the bottom of the 3-in. pipe prevents the solution from coming in contact with the metal.

RESULTS OF OPERATION

The plant was put in use about October, 1913. The success of its operation from the standpoint of the public is best expressed by a remark of the superintendent of sewers, James W. Reid, who stated to the writer a short time ago that the people have forgotten its existence. When the flow through the plant is small, the greater part of the sewage passes through the discharge tubes first reached, but, for average flows, the discharge through all of the tubes is quite uniform. The wind-guard pipes perform their service well, as the column of falling sewage is deflected only slightly from the center of the disk, even when the wind is quite high.

The method of disk adjustment has proven thoroughly satisfactory. A disk tested for center with the falling column of sewage can be quickly shifted to its proper position and rigidly held there.

Little trouble has been experienced from clogging of the discharge tubes. Floating patches of scum, which forms in considerable quantity, even on the fresh sewage from the separation tank, are held back by the coarse screen in the control chamber. Scum forming in the overhead channels is removed each day with a perforated dipper. Any scum formed can be easily removed in its entirety, whereas only small bits can be taken from spray nozzles, leaving the large patches undisturbed in the distributing pipes to furnish more small pieces in a short time.

There never has been the slightest sign of clogging in the filter bed.

Inspection of the central well of the final sedimentation basin a few weeks after the plant had been put in operation showed the water confined within it to be very dark and quite similar in appearance to septic-tank liquor, proving that the sediment from the sprinkling-filter effluent was undergoing decomposition. On recent examination of the final sedimentation basin, the whole content of the settling chamber proper was found to be somewhat darker than should reasonably have been expected, with numerous gas bubbles rising through the liquid. This condition was found to be due to neglect of the final sedimentation basin, the sludge having been allowed to collect until it had filled not only the central well but also a large part of the settling chamber. In fact, no attempt had been made to pump the sediment into the decomposing chamber of the separation tank. If necessary, this sediment will be stirred with a heavy pole to thin it sufficiently for pumping.

The disinfection plant has never been put into successful operation. At the time it was constructed, a test with water showed that neither of the storage reservoirs was airtight. An air pump has since been furnished the city for use in discovering the air leaks, but no effort has been made to use it. The stoneware valves were wholly unsatisfactory, as both leaked badly. It is probable that disinfection will not form a part of the treating process until complaints of property owners below the plant stir the city to action. This indifference is largely due to the anticipated inconvenience of mixing the solution each day and the desire to avoid the expense of the hypochlorite of lime.

Thus far the screenings have been thrown upon the ground near the plant without any precaution to prevent a nuisance. The dried sludge from the separation tank is practically odorless. It is removed from the sludge bed in a wheelbarrow and deposited at convenient points.

The plant is visited once each day by the superintendent of sewers or a laborer in his employ for the purpose of cleaning the screens, removing any scum collecting in the distributing channels, and unstopping any clogged discharge tubes. The average time spent at the plant each day is about 45 minutes.

All of the precautions taken and the expense incurred in the construction of the plant in the effort to prevent a nuisance from its operation have been amply justified. There is not the slightest odor observable from the screen chamber or the separation tank. An earthy smell, characteristic of slightly stale sewage, can be detected when within about 20 ft. of the sprinkling filter and on the windward side, but it is so faint even at this short distance that there is no possibility of its being noticed beyond the limits of the property purchased by the city for the location of the plant.

*In order to disinfect settled sewage when the height of the water in Tidwell Creek prevents the operation of the sprinkling filter, a ¾-in. galvanized-iron pipe is used to conduct the solution to a manhole on the bypass sewer. This pipe terminates at the orifice tanks in a short horizontal piece, on the end of which is a 1½x¾-in. reducing elbow. By swinging this elbow under the end of the discharge tube of either orifice tank the solution can be intercepted.

The Jitney Bus*

BY C. I. PALM†

There probably was never an adverse movement against public utilities which has spread as rapidly through the country as the "jitney" bus craze. In less than four months, it completed a sweep of the country, invading cities in nearly every state.

In November, 1914, some owners of second-hand automobiles in Los Angeles, being out of employment, thought they could pick up a few "jitneys," or nickels, by carrying players to the various moving-picture establishments. The owners of these small cars had read about the big and apparently highly profitable business being done by

*From a paper read before the Iowa Street and Interurban Railway Association

†Superintendent, Omaha and Council Bluffs Street Ry. Co.

a "$500,000 motor-coach company" operating between Los Angeles and neighboring villages, but at rates of fare considerably in excess of 5c. This latter company, by the way, went into the hands of receivers last December. Its busses were taken to satisfy a mortgage and were transferred to San Francisco, to be operated as jitneys during the exposition.

Jitneys started operating in Omaha early in February. At this writing we have 54 jitney busses. Four of these are seven-passenger cars, six are trucks seating from 9 to 14 passengers, and the remainder are five-passenger cars, mostly Fords.

No ordinance looking toward their regulation has as yet been passed by the municipal authorities, although improvement clubs and other civic bodies have urged that such an ordinance be passed. The jitneys in Omaha already have had a number of accidents (one fatal), and as they grow in number the percentage of accidents will be larger. The operators are organized as the Omaha Jitney Bus Men's Association, with a membership of 75.

A taxicab company in New York City, operating Ford cars, estimates that its expenses per car mile amount to 11.51c. This figure is based on the assumption that the car would run 15,000 mi. annually. If it makes 25,000 mi. in a year, this cost would be 10.7c. In this cost, however, there are a few items, such as advertising and meter expenses, which the jitney operator does not have. Eliminating these makes the cost about 10c.; so these cars have to earn at least 10c. per car mile to pay expenses.

Let's figure it out ourselves: The present price of a Ford machine is $525, delivered at Omaha. Assuming it can run 50,000 mi. before it is ready to be scrapped, there will be a charge for depreciation of 1.05c. per mi. If the driver is paid $3 per day and the car runs 100 mi. per day, the cost of this item will be 3c. per mi. Gasoline at 12c. per gal., and 18 mi. per gal., figures 0.67c. per mi. Lubricating oil at 20c. per quart, 50 mi. per quart, makes this cost 0.4c. per mi. A set of tires cost about $40 and the average life is not more than 4000 mi.; the per mile cost figures 1c. Repairs, cleaning, storage and miscellaneous supplies will easily amount to another cent per mile.

Summarizing these items, we have:

Depreciation	1.05c. per mile
Wages of operator	3.00c. per mile
Gasoline	0.67c. per mile
Lubricating oil	0.40c. per mile
Tires	1.00c. per mile
Repairs, storage, supplies, etc.	1.00c. per mile
Total	7.12c. per mile

Dividing this by 4 (seating capacity) equals 1.78c., the cost per passenger-seat-mile.

The above estimate does not include anything for injuries and damages resulting from accidents, which are bound to happen. Nor is any allowance made for license fee, taxes, interest on the investment or insurance, all of which the taxicab company has to take into consideration, and so should the jitney owner.

But the argument is advanced that most of the jitney men buy cheap second-hand cars for this service, consequently the charges for depreciation and interest are low. This may be true, but it must be remembered that the item of repairs on such a car is much greater than on a new car, and this difference will, no doubt, offset the difference in depreciation and interest between a second-hand car and a new one.

It is apparent, therefore, that it costs at least 7c. per mi. to operate the cheapest kind of a jitney. What, then, can it earn? The length of the average trip is about 2½ mi. one way, or 5 mi. for the round trip. This is the case in Omaha. Assuming that the jitney averages three passengers per trip, which is high, the receipts will be 30c. for a round trip. Three round trips per hour allow a speed of 15 mi. per hr., which is about right, taking stops into consideration. Therefore, the earnings of a jitney would amount to 90c. per hr. Its operating expenses at 7c. per mi. amount to $1.05 per hr.

It would seem that anyone, before venturing into this speculative business, would investigate the auto-bus of the larger type and its cost of operation, which type of bus has been operating both in this country and in Europe for a number of years.

Authoritative figures on the cost of this latter type of bus operation in this country have been stated to be 24.51c. per car mile for a double-deck bus seating 42 passengers. This is approximately 0.6c. per passenger-mile. To equal this passenger cost, the small jitney bus, carrying only four passengers, will have to operate at less than one-tenth the cost of the auto-bus seating 42 passengers.

The majority of the ordinances and statutes that have been passed or proposed by authorities having the jitney problem to deal with require the filing of an indemnity bond by the operator for the protection of the public in case of accident. The number of jitney accidents occurring daily is awakening the public to a realization of the importance of this protection. The patron of the jitney is not alone in danger of accident, but pedestrians are as well. The danger of the situation has been aptly expressed by a person on the Pacific Coast, where the craze has become a real menace. He remarked that "there are only two classes of people downtown nowadays—the quick and the dead."

Opposition to the jitneys and petitions for their regulation have not come from the street-railway companies, although they have, of course, protested. They have come from the business men, improvement clubs and other civic bodies, who have realized the menace and irresponsibility of the whole proposition.

But the craze has reached its height and is now waning. The small-car owner, if he has not already done so, will soon find out that he cannot carry a passenger for 5c. and make a profit. Investigation has shown that very few of the men who operated jitneys a few months ago are still in the business. The few who are staying in the game are doing so because it is the only employment they can find; they are satisfied with the income remaining from the nickels collected less the cost of gasoline and minor repairs. When their machines are worn out, or if they should ever meet with a bad accident, they realize the jig is up. The proposition is, therefore, largely a product of unemployment and cannot continue to exist for any length of time. That it cannot do so under proper regulation is further evidenced by a statement made by the jitney owners in Los Angeles that the passengers on the running board are the profits of the business.

I believe there is a field for the auto-bus, but not as a 5c. proposition. Its mission is rapid transit *de luxe*. A no small proportion of the population in the larger cities would, without doubt, be glad to pay a higher rate of fare than that charged by the street railways for a guaranteed seat in a comfortable and uptodate vehicle operating at a rapid but safe rate of speed.

Proposed Idaho State Highways

The Idaho State Highway Commission, established in 1913, has the problem of building roads in one of the most sparsely settled and mountainous sections of the country.

Most of the time of the engineering staff thus far has been directed to surveying and studying certain trunk-line highways. Those selected are known as the Idaho-Pacific Highway, extending from the Idaho-Utah boundary near Fish Haven, Bear Lake County, through Pocatello, Boise, Lewiston, Cœur d'Alene; the North Pacific Highway, branching from the Idaho-Pacific Highway at a point near the Idaho-Washington boundary and extending through Mullan to a point on the Idaho-Montana boundary; the Idaho-Utah Highway, extending from the Idaho-Pacific Highway through Bannock and Oneida Counties to the Idaho-Utah boundary; the Yellowstone Park Highway, from Pocatello to the western entrance of Yellowstone National Park; the Idaho-Montana Highway, extending from Idaho Falls northerly to a point on the Idaho-Montana boundary; and the Sawtooth Park Highway, extending from Twin Falls via Shoshone Falls to Hailey in Blaine County. These highways comprise a total length of about 1300 miles.

To those familiar with the Idaho country, the problem of locating these highways with 8% maximum grades is appreciated. The proposed location of the Idaho-Pacific Highway, 800 mi. long, from the Utah to the Washington State line, has no grade exceeding 5%, although it is questioned if this is justifiable. The accompanying illustration shows graphically the amount of development necessary to maintain this 5% grade near Lewiston, on the Idaho-Washington boundary line.

The Chief Engineer of the Idaho State Highway Commission is E. St. C. Smith.

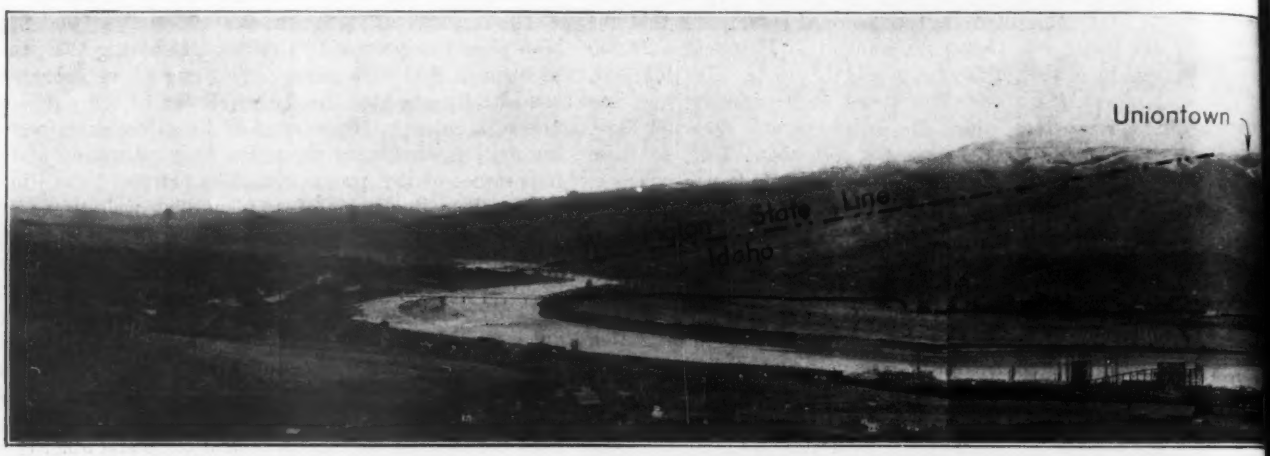
Automatic Signals for the Elevated Railways in New York City have been ordered by the Public Service Commission for the First District of New York. The system must be designed to prevent collisions and permit operation of tracks to maximum capacity. A trial installation is ordered placed in operation by Oct. 1, 1915. The existing signals of the elevated roads are limited to automatic block equipment at curves and manual semaphores at interlocking switch-control points.

Concrete Pavement Damaged by a Fire

By C. C. Wiley*

On the morning of Mar. 17, 1915, the City of Champaign, Ill., was visited by a serious fire, which threatened to sweep over a large area in the business district. It originated in a four-story office and store building which, together with a narrow two-story building, occupied half a block along Neil St., one of the principal business streets. Both buildings were completely destroyed and the flames, carried by the wind, crossed the street and practically demolished the opposite block of two-story mercantile buildings.

The intervening pavement is 44 ft. in width between curbs. It consists of an 8 ft. strip of vitrified brick along the single car track in the center, flanked on each side by an 18-ft. strip of concrete. The brick section is of a fair quality of vitrified block with asphalt (Sarco) joint filler. The concrete sections are 6½ in. in thickness and are composed of a 1:2:3 mix of excellent sand and gravel, reinforced with a single layer of No. 29 triangular-mesh reinforcement. Expansion joints of Elastite armored with Baker plates are provided about 26 ft. apart.



Development of a 5% Grade on the Proposed Loca-
(Total rise is 1900 ft. in

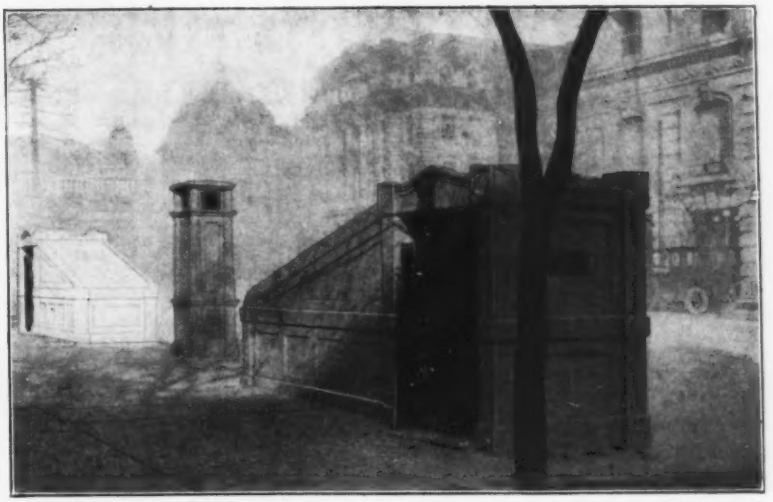
The concrete was laid early in October, 1914, and although mixed somewhat too wet for the character of the materials and the climatic conditions, it was finished fairly well by means of long-handle floats. In other respects the workmanship was of at least average quality. However, as the weather was cool the concrete was neither sprinkled nor covered while curing, and it was opened to traffic after only 25 days. It came through a moderately severe winter in good shape except that it shows some wear and a few places have shown a tendency to scale, probably owing to the excessive wetness of the concrete and the method of finishing. The sections in the path of the fire, however, were apparently in excellent condition.

During the fire a hundred feet or more of the pavement was subjected to intense heat as the flames were carried

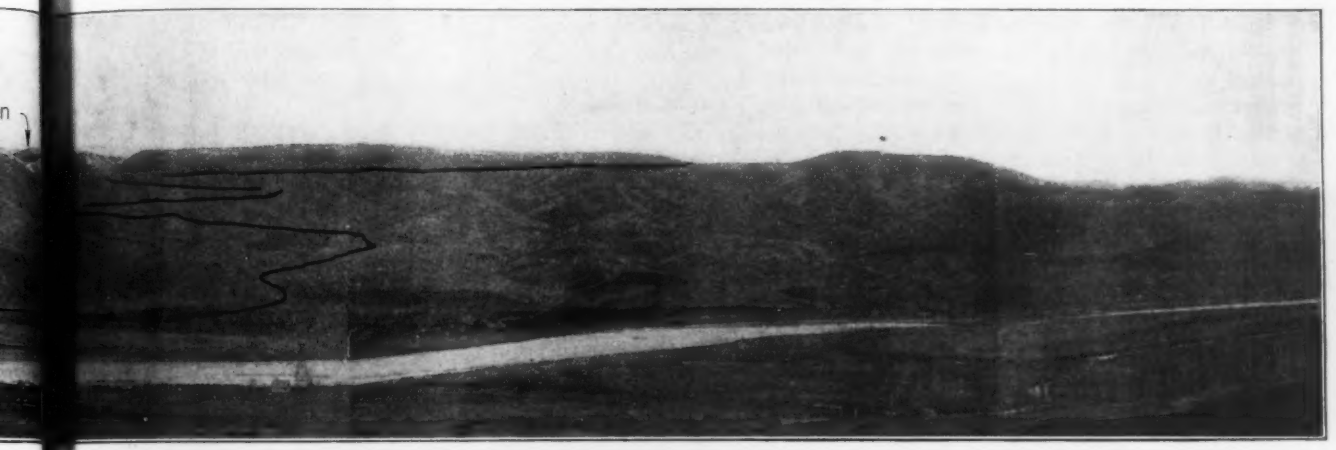
*Associate in Civil Engineering, University of Illinois, Urbana, Ill.

down onto the surface by the wind. In fact, it became so hot that the asphalt filler boiled out of the joints. After nearly an hour the walls of the four-story building fell, burying the pavement under a mass of brick and stone, the heavy blocks of red sandstone from the coping striking about the middle of the concrete strip. Two weeks or more elapsed before the débris was sufficiently cleared away for the effect of the impact to be seen.

The surface of the pavement was quite seriously damaged by the heat and shock. The mortar and smaller pebbles were crushed to powder, and many of the larger stones were shattered and displaced. The surface was gouged out over a considerable area for a depth of perhaps an inch or more, and in addition many spots were pitted



ENTRANCES AND AIR SHAFT OF PUBLIC-COMFORT STATION, SALT LAKE CITY, UTAH



TION OF THE IDAHO-PACIFIC HIGHWAY NEAR LEWISTON
a distance of 10 miles)

to almost as great a depth. None of the débris reached the section of brick surface, so no direct comparison can be made. On the cross street, however, a sand-filled brick pavement that was subjected to the same impact, but not to the same heat, is practically undamaged. A very few spalled or cracked brick seem to be the only evidences of any abnormal treatment.

As yet no attempt has been made to repair the damaged concrete surface, and apparently it has not been decided whether to attempt surface patching or to relay the damaged sections.

Public-Comfort Station Built in Salt Lake City, Utah

BY SYLVESTER Q. CANNON*

An underground public-comfort station, designed to accommodate one hundred persons per hour, has recently been completed by the City Engineering Department of Salt Lake City at a cost of about $13,000. It is located beneath the wide sidewalk near the intersection of South Temple and Main St., and has sheltered entrances near the curb, as shown in the accompanying view.

The station has two sections, one for men and one for women. Its inside dimensions are 17x21 ft. for the men's and 17x23 ft. for the women's section, besides which there are a small fan room and closet and an alley between the two sections and also another alley at the rear. The overall dimensions are 21½x50 ft., not including the covered entries to the stairways.

There are five water-closets, two washbasins with hot and cold water, a fountain and a rest room in each section, and a babies' room in the women's section.

The structural work is of reinforced concrete. The floors are set with white tile and the partitions are of white marble. The walls of the rest rooms and also of the stall rooms to the top of the stalls are finished with glazed tile. All the ceilings and the portion of the stall-room walls above the tops of the stalls are finished with Keen's cement, pointed with white oil and coated with enamel.

Besides the sidewalk lights in the ceiling, electric lights are provided. The station is heated by steam. A motor-driven fan changes the air every three minutes.

*City Engineer, Salt Lake City, Utah.

Field and Office

A Builder's Mast Crane which Proved a Success

When Willing Bros., contractors, of Bellevue, Ohio, were awarded the contract to build the Children's Home at Norwalk, they set about to devise the most economical method to handle material into place. The result was that they designed a revolving mast crane with cantilever boom; the Klein Iron Works Co., of Bellevue, built it. Two trolleys operating on the cantilever boom handled 2-ton loads out any distance up to 64 ft. from the mast.

Fig. 1 shows the crane as used in building the Children's Home; it is carrying an I-beam on the left-hand trolley and a tray of bricks on the right. The success of the crane on this work was marked. The stonework was handled at about one-third of the estimated cost. W. L. Willing, who invented the crane and has applied for patent on it, says that on a long or strung-out building it beats anything else used in that class of work. It covers all the space in the most direct way. There are also some incidental advantages; for example, in placing concrete in beams or floor the bottom door of the concrete bucket is opened only part way, and by moving the trolley along the boom the concrete is spread as it runs out of the bucket.

The crane, as built, has power hoist and power traverse of the trolleys, but hand rotation (the next crane of this style will probably have power rotation). The mast is 56 ft. high. The boom is 131 ft. long out to out, each arm being 64 ft. long and the length of the part through the mast 3 ft. In construction, the mast is a narrow compact tower made of angle posts and angle and bar latticing. The boom is made up of a web plate 8 in. deep and two angles, one on either side near the lower edge of the plate, on which the trolley wheels travel. The upper edge of the boom is stiffened by horizontal lattice.

The trolleys running on the boom, one on each arm, are moved back and forth by ropes that pass over horizontal drums in the mast just at the level of the boom. These drums are driven by worm gear from a direct-and-reverse belt drive, which gets its power near the foot of the mast by chain and gear drive from the main motor, a 5-hp. machine (see Fig. 2). The drum near the base of the mast, seen to the right in Fig. 2, handles the hoist rope of one trolley; a similar drum back of the mast takes care of the other trolley.

The load can be hoisted at the rate of about 75 ft. per min. In lowering, the clutch is thrown out and a brake is applied by a foot lever in the crow's-nest. The brake is sensitive enough to give perfect control of the load, which is of particular advantage in setting stone.

The mast is rotated by a handwheel located at the crow's-nest, actuating a vertical shaft (at the back of the mast in Fig. 2) which carries a pinion meshing with a fixed gear at the base of the crane.

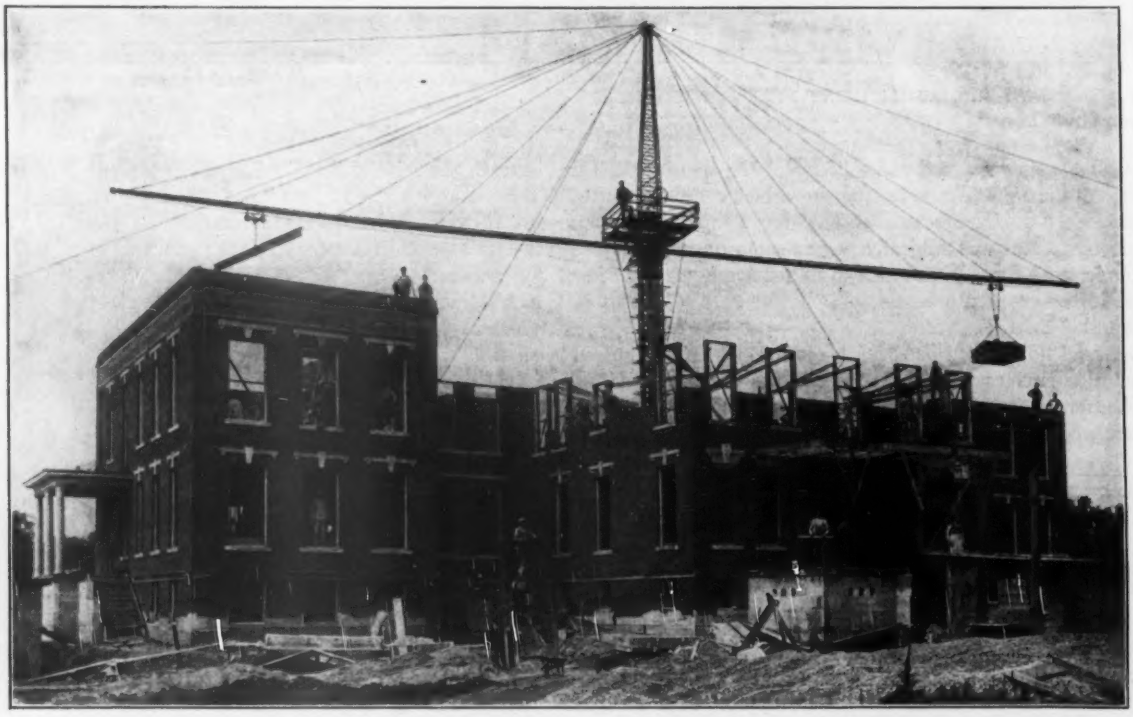
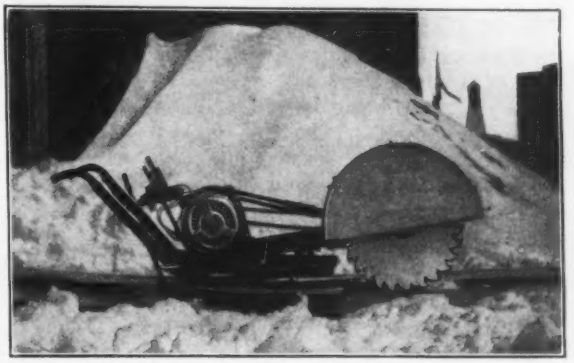


FIG. 1. WILLING BROS.' MAST CRANE, WORKING ON MASONRY OF CHILDREN'S HOME, NORWALK, O.

Motor-Driven Ice Saw

The circular-saw outfit shown in the accompanying view was developed by C. A. Edwards, civil engineer and irrigation superintendent, Gothenburg, Neb., to reduce the cost of harvesting ice and to improve the sanitary conditions of the work. Before this machine was secured two markers and four plows were usually employed, with a team and two men for each tool. With the old methods there were many broken and unclean cakes. In the season just past two men with this saw did the work of the 6 teams and 12 men, in 21 hours, cutting 105 carloads of 33 tons each.

A 28-in. circular saw and a 20-hp. motor were fastened to a rigid steel frame and connected by a chain belt. The saw frame was mounted, nearly balanced, on a sled with runners 44 in. apart and arranged to follow the previous saw-cut as a guide. The saw could easily be raised or lowered. The upper half of the disk was covered with a shield to protect the operators from flying ice chips. Electric energy was taken from a power house 2000 ft. away; on the lake side a three-phase bank of transformers cut down the potential from 2300 to 220 volts. From the transformers 900 ft. of three-wire line ran out and this was moved once a day. The circuit was continued with 500 ft. of flexible cable attached to the plow and moving on the ice as the outfit traveled along. One man guided the saw and the other handled the cable. There was a switch near the guide handles to turn the motor off or on, and two lamps to show when the voltage was on the supply line and to give light at night. The saw made a cut 3/8 in. wide and 10 or 12 in. deep. If the teeth were sharp the rig would feed and propel itself, but could easily be started and stopped.



Motor-Driven Ice Saw

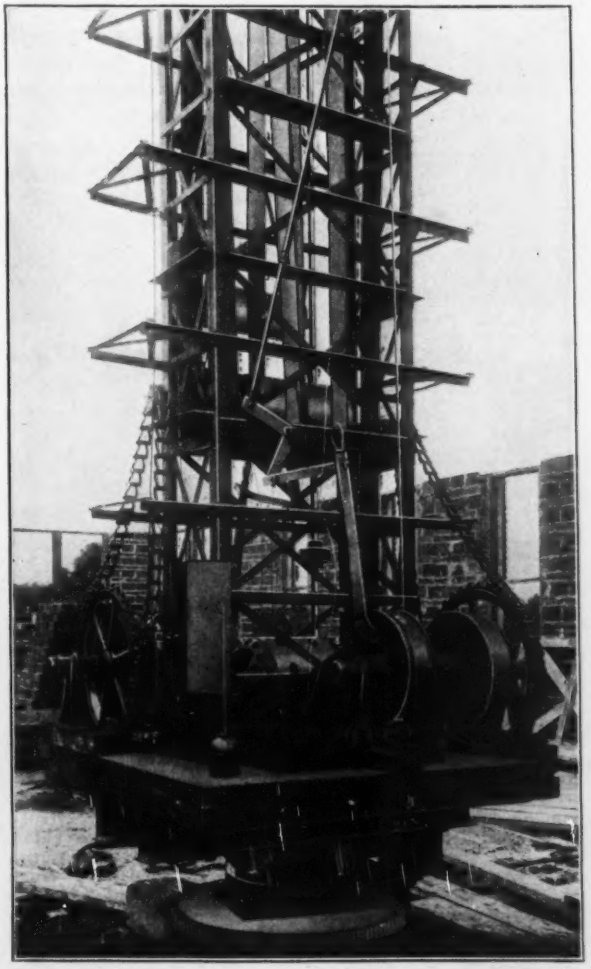


Fig. 2. Base of Willing Crane, with Swinging Gear, Motor, Hoist Drum, and Travel Drive

Coiling a Long Tape by Figure-Eight Method

The usual one-man method of doing up a short tape and throwing it into a circle is not applicable to tapes much over 2 chains long, owing to fatigue of the chainman's hand in holding the loops. Nor is it always practicable to do the tape up on a reel, since reels are usually too heavy or cumbersome for the chainmen to carry with them and are usually somewhere else in the wagon when wanted.

L. E. Wilkes, U. S. Surveyor, uses a method for any long tape which for lack of a better name may be called the two-man method, since two men are required to make use of it. One man does the work, the other simply extends his hands close together, fingers up and palms toward him, to hold the tape. The man coiling up the tape takes the front end between the thumb and first finger of his left hand, with palm up, the tape extending back through his hand. Then he runs his right hand, palm down, back along the tape, takes a convenient length, grasps the tape between thumb and first finger, and passes it over the extended fingers of the second man's hands and brings it back through his left hand in the opposite direction to which the front end extended, thus forming a large, flat figure eight. This operation is repeated to the end of the tape. It is to be noted that the tape where it crosses itself must lie flat. To accomplish this it must be turned over in passing from one man's hand to the others; then when the tension is relaxed, the figure eight will fly into a circle. The movement of turning the tape is accomplished by a simple twist of the wrist. This method of doing up the tape is exceedingly rapid when understood. The halves of the figure eight should be tied separately and then allowed to spring into the form of a circle. One man can undo the tape, and no tangles result, three months' use of this method having failed to produce a single tangle.—H. N. Bradstreet, U. S. Transitman, Phœnix, Ariz.

Gusset-Plate Diagram

By W. M. Eliot*

In detailing ordinary roof trusses much time is spent in making large-scale drawings to determine the distance from the working point to the ends of adjacent angles and the size of gusset-plate required. The accompanying diagrams (Fig. 1), when drawn to full size on heavy paper, will take the place of these layouts at a great saving in time. Their use will be illustrated by the solution of the details of joint *A* of Fig. 3.

Place the scale on the diagram so that the gage line of the web angles coincides with the required bevel of 12 on $10\frac{9}{16}$ when the edge of the angle on the scale (found at $b = 1\frac{3}{8}$ in.) lies on the clearance line on the diagram

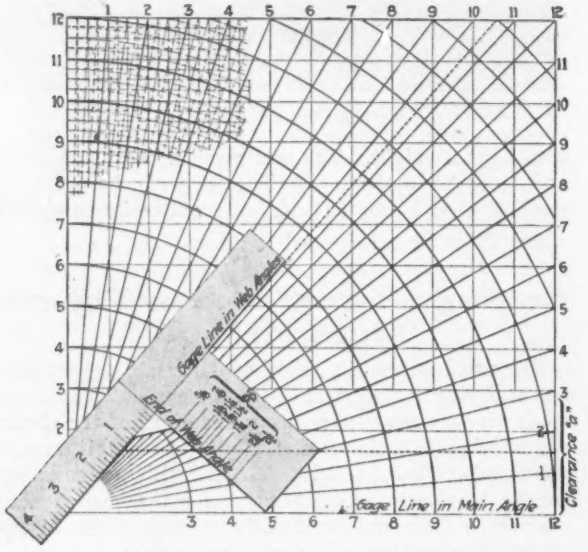


Fig. 1. Gusset-Plate Diagram

(found at $a = 1\frac{1}{2}$ in.). Read $3\frac{1}{4}$ in. on the gage line, which is the distance from working point to end of angle. The distance to the end of the other member is found in the same way.

Setting the scale in correct position is rendered easy by sticking pins in the diagram at the working point and

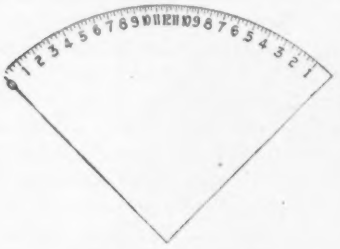


Fig. 2. Bevel Protractor

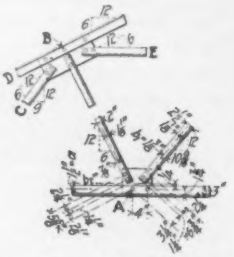


Fig. 3. Roof-Truss Joint

at the given bevel, and moving the gage line of the scale along the pins.

The distance to the last rivet in the web angle is obtained by simple addition. By locating $6\frac{3}{4}$ in. on the bevel 12 on $10\frac{9}{16}$, on the diagram, the distances along and at right angles to the gage line of the main angles may be read. The coördinates for the last rivet of the other angle are found similarly. By taking the largest distance out, adding the coördinates to the right and left, and allowing for edge distance, the size of gusset-plate is secured.

When the main member is inclined and all bevels are given with respect to the horizontal, the bevel of the web member with respect to the chord may be obtained by locating the two given bevels on the diagram and reading the angle between them in terms of bevels with a bevel protractor such as shown in reduced copy in Fig. 2. The solution then becomes the same as for a horizontal main member.

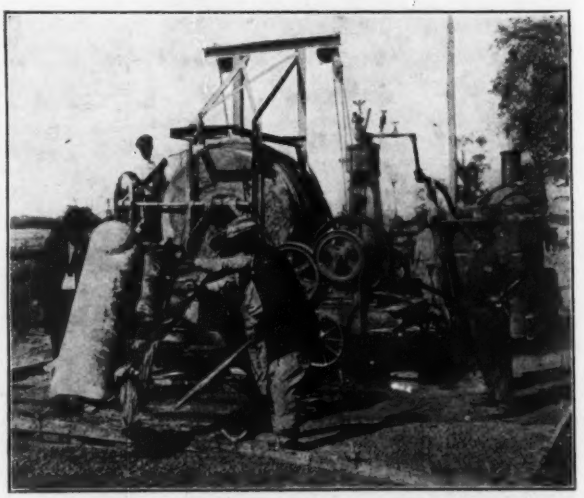
Any draftsman can make the diagram on heavy drawing paper in a short time. As full-size scale is used, accurate results should be obtained.

*713 North Lancaster Ave., Dallas, Tex.

Adapting a Concrete Mixer to Road Work

The great development of the last few years in the use of concrete for road building has brought out many concrete mixers especially designed for road work—paving mixers or "pavers." With a little ingenuity, however, almost any mixer may be made over into a traveling unit for road work.

The accompanying view showing the construction of the Oxford Pike service-test road of the Bureau of Highways, Philadelphia, Penn., illustrates how a Ransome side-discharge mixer was made into a "paver." Wm. H.



A Side-Discharge Concrete Mixer Converted into a "Paver"

Connell, Chief of the Bureau of Highways, describes the changes thus:

> In order to adapt it to road use and to facilitate quick moving, the expedient of mounting the mixer upon a truck was resorted to. The boiler of the machine was dismounted, this preserving the balance of the apparatus as mounted upon the truck and reducing the dead-load to be moved. The truck ran upon railway tracks built in 8-ft. sections, the 8-ft. rails being bolted to 3-in. flat planks and moved as one unit. The units were bolted together with fishplates as the truck was moved from one section to another. Motive power was furnished by a tandem steam roller, not only supplying the steam for the mixer engine but also acting as a tractor in moving the plant.

With the previously cited arrangement on an average 300 lin.ft. of concrete pavement per day was laid, 18 ft. wide, with an average depth of concrete of 6¼ in.

Cheap and Effective Backfilling with a Road Roller

For rapid, cheap and effective backfilling of trenches the Los Angeles City Water Department has utilized its steam road roller, rigged up to what the force calls a "pusher," the contrivance being capable of doing three times the amount of work that can be accomplished by an average scraper gang, at one-half the expense. The device is shown in the illustration. It consists of a piece of Oregon pine, 2x12-in. by 3 ft. long, shod with iron, well braced and bolted to a 6x6 Oregon pine beam, 16-ft. long, attached after the manner of a wagon tongue to the front of a 7-ton 10-hp. Kelly-Springfield road roller. The beam is supported by tackle from a mast of 4x4 Oregon pine fastened on a hinge to the frame of the roller and held in place by wire guys.

This machine was used on backfilling 5 miles of ditch excavated for a 40-in. riveted steel main, the trench being 5 ft. deep and 5½ ft. wide. The spoil bank, which was of reasonable dry earth, had a base of approximately 12 ft. and a height of about 6 ft. After the men became accustomed to the use of the equipment, with every forward trip of the roller a cubic yard of fill went into the trench. Not only was the machine satisfactory from the volume of earth moved, but it was found that the roller could back up to a freeway, go forward again and have its load in the ditch in the time that a two-horse fresno was getting into position for its load.

PUSHER ATTACHED TO STEAM ROLLER, BACKFILLING A PIPE TRENCH

The machine on this ditch in an 8-hr. day did an average of 450 lin.ft. of backfill, the expense being one man to guide the "pusher" or plowshare at $2.50; a second man to raise and lower the tongue by means of the tackle at $2.50; a steam engineer for the roller at $3.50; and 800 lb. of coal at $13.50 per ton—a total of $13.90. (This rate would be considerably lowered in a locality where cheap coal is to be had.) On the same work before the contrivance was put in service two two-horse scrapers and drivers at $3.50 each and one man as helper at $2.50 did 150 lin.ft.

In working out the final form it was found that the length of tongue should be nearly as long as the base of the spoil bank to be moved; also that the device must be very strongly built. Two conditions necessary for good work are a fairly level ground with no obstructions in the way and sufficient room in the street for the length of roller and tongue to get at the outer edge of the bank. It is quite possible to take the bank at an angle, but the best results are accomplished when the bank is hit squarely. Also, the use of the machine involves stopping of traffic in the block in which it is working.

The device was worked out by Thomas Brooks, assistant superintendent.—BURT. A. HEINLY, 645 S. Olive St., Los Angeles, Calif.

Average Cost of Concrete Pavements Laid in 1914

The Association of American Portland Cement Manufacturers, as many of our readers know, maintains a corps of inspectors and engineers whose services are furnished to cities, counties and states constructing concrete highways to the end that the work may be carried out in such a manner as to produce a satisfactory and durable roadway. In a paper entitled "Cost and Economy of Cement-Concrete Pavements," read at the Cornell "Good Roads Week" in February, H. E. Hilts, engineer of the Association, presented some valuable figures, showing the average cost of 1,500,000 sq.yd. of concrete paving laid on roads and streets in 1914 under the inspection of the association. These figures represent work performed on 95 different contracts in 20 different states.

The accompanying table, which we take from Mr. Hilts' paper as published in the *Cornell Civil Engineer* for March-April, gives an average cost per square yard for the 1,500,000 sq.yd. of pavement of $1.058. It should be noted that this is actual cost and does not include the contractor's overhead charges and profits.

In studying the table it should be noted that figures from states where only a small amount of concrete pavement was laid are of comparatively little importance, as special conditions may have caused certain items in the cost to be abnormally low or abnormally high. The figures, from such states as New York, with 730,000 sq.yd. laid, Connecticut, with 200,000 sq.yd. laid, and Iowa, with 90,000 sq.yd. laid, represent enough work to make the

itemized figures valuable. It is of interest to notice, as an illustration of the efficiency with which the building of concrete roads is now performed, that of the total cost of the paving 70% is the cost of materials, leaving only 30%,

AVERAGE COST OF CONCRETE PAVEMENTS LAID ON 95 ROAD CONTRACTS IN 1914

State	Labor	Teaming	Mechanical Equipment	Materials	Miscellaneous	Total	No. Sq.Yd.
Connecticut	$0.181	$0.142	$0.046	$0.752	$0.010	$1.088	200,056
Florida	.108	.028	.003	.838	.014	.991	13,000
Indiana	.134	.130	.044	.777	.022	1.017	89,794
Illinois	.136	.101	.033	.528	.016	.814	17,970
Iowa	.213	.084	.028	.789	.036	1.116	90,364
Kansas	.396	....	.026	.819	.007	1.248	1,503
Kentucky	.146	.214	.020	.477	.018	.875	4,144
Maryland	.111	.127	.065	.716	.057	1.077	36,211
Massachusetts	.195	.104	.039	.600	....	.947	26,200
Michigan	.222	.032	.021	.655	.050	1.028	92,217
Mississippi	.261	.085	.150	.642	.004	1.232	12,031
Missouri	.209	.133	.027	.656	.059	1.045	22,376
New Hampshire	.126	.097	.032	.907	....	1.152	6,066
New Jersey	.179	.089	.016	.797	....	1.091	8,356
New York	.156	.085	.055	.666	.050	1.027	730,782
Ohio	.181	.091	.067	.642	.009	.946	58,968
Pennsylvania	.279	.091	.031	.742	....	1.269	10,072
Texas	.316	.117	.007	.668	.016	1.123	6,640
West Virginia	.098	....	.060	.912	....	1.070	10,045
Wisconsin	.188	.110	.045	.682	.023	1.033	61,879
Grand average	$0.183	$0.104	$0.040	$0.717	$0.046	$1.058*	1,498,883

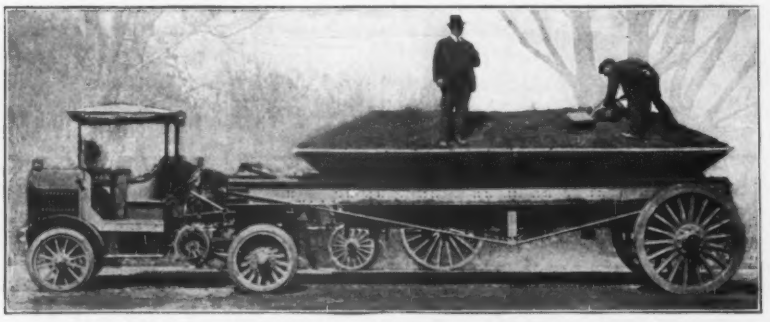
*Exclusive of profits and overhead charge.

or a little over 30c. per sq.yd., for the entire cost of the labor force, the hauling of the materials and the use of the mechanical equipment on the job.

A Large Double Dump-Wagon

The very large dump-wagon shown in the accompanying view is part of the equipment of the Metropolitan Street Ry., of Kansas City, Mo., being used with a tractor for handling paving and construction material. The wagon has only two wheels, its forward end being supported by a kingpin connection on the rear of the four-wheeled tractor.

The dump-wagon is of a special type which (with four wheels for horse traction) has been built for some time by the Bull Dog Manufacturing Co., Kansas City, Mo. The body is divided into two sections by a transverse partition. Each section has a central ridge and side doors, so that either one or both sides of the hopper can be dumped and the two sections can be operated independently. The dumping mechanism is controlled by levers near the driver's seat. The hopper body (removable) is of steel, mounted on a timber bed. It is 7 ft. wide at the top, has a capacity of 10 cu.yd. and is rated for 12½ tons. The total height of the wagon is 6 ft. 6 in., and the body is carried high enough to give a clearance of 30 in. under the doors when dumping. The wheels are 4 ft. 9 in. in diameter, with 8-in. steel tires 1¼ in. thick. The wheelbase is 19 ft. 3 in. from tractor axle to trailer axle, but with the flexibility due to the king-bolt connection the outfit can be turned readily in a limited space.

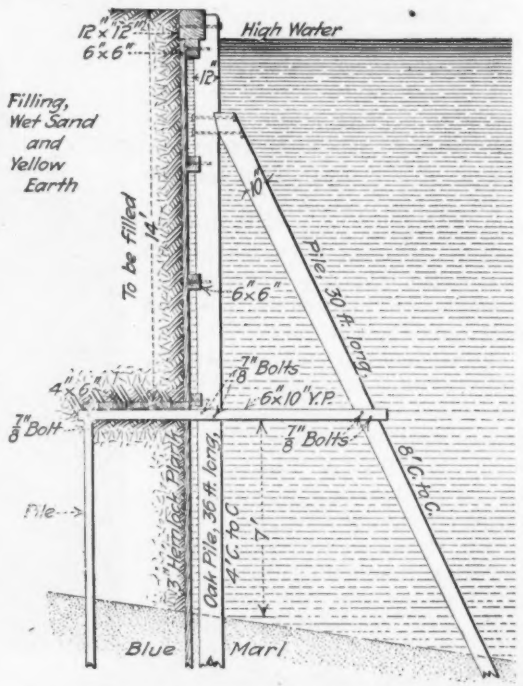


DOUBLE DUMP-WAGON OF METROPOLITAN STREET RY., KANSAS CITY

The weight of wagon empty (with two wheels) is 8500 lb. When loaded, about 65% of the weight is carried by the wagon wheels and 35% by the rear of the tractor. A 5-yd. wagon with single hopper weighs about 5200 lb.

A Bulkhead to Retain Soft Fill

The bulkhead shown in section in the accompanying sketch was devised by F. J. Barrett, 176 Federal St., Boston, Mass., to retain a 21-ft. hydraulic fill which



TIMBER BULKHEAD WITH RELIEVING PLATFORM

had exerted so great a pressure as to push out of line an ordinary braced bulkhead already erected. The original structure was composed of wooden piles driven 4 ft. c. to c. into a solid clay bottom, with interior 3-in. sheetpiling driven against 6x6-in. waling pieces, the whole being braced with batter piles fastened, as shown in the cut, to every other vertical pile.

As soon as filling commenced, the bulkhead began to move from 1 to 3 ft. out of line. Anchor piles were then driven as deadmen inside the wall and an attempt was made to maintain verticality by tying back to these deadmen with ⅞-in. galvanized cable. This was ineffective, for the movement continued.

At this juncture Mr. Barrett decided to revise that part of the wall behind which the fill had not been attempted by building a horizontal supporting platform of the type shown in the sketch. This platform is made up of 4x6-in. second-hand sticks laid as a floor on horizontal 6x10-in. sticks, which were slid through openings cut in the sheetpiles, notched into and spiked to the vertical

and batter piles and supported on the inside on short piles, as shown. The fill made behind the structure so built was held in perfect alignment, and the remainder of the wall was completed in similar fashion. The bulkhead has stood for two years without movement.

The intention was to design the platform of such area and at such a depth as to provide a downward loading sufficient to overbalance the thrust of the fill. The rigidity of the structure, furthermore, was increased by the stiffening effect of the cross-beams spiked to both sets of piles.

Comparative Costs of Rotary and Standard Drilling

The results of a number of recent tests to determine the relative economy of rotary and standard drilling in the oil-fields of California were compiled by M. L. Requa, of San Francisco, and presented in the form of a brief paper at the annual meeting of the American Institute of Mining Engineers in New York City in February. A summary of these results follows:

Average standard time	= 88.3 days
Average rotary time	= 60.3 days
In favor of rotary	= 28 days
Standard, average cost per ft.	= $3.42
Rotary, average cost per ft.	= 3.24
In favor of rotary	= 0.18
Production assumed at 250 bbl. per day, 28 days	= 5600 bbl.
In favor of rotary	
Oil, 5600 bbl. at 50c.	= $2800
Drilling 1955 ft. at 18c.	= 352
Total	= $3152

It will be noted from the foregoing figures that the rotary drill not only performed the work in quicker time but at lower cost per foot. Similar tests made in 1910 by the Nevada Petroleum Co. seemed to prove that the reverse was true. Since then, however, improvements in rotary drilling machines have removed many of the objections to this form of drilling. One of the most important of these improvements was the invention of the cone bit for rock work. This bit is fully described by the inventor in another part of this issue.

Burn Your Dynamite Cases

Dynamite, when spread out on the ground so as to expose as large a surface as possible and ignited, generally burns up peacefully without exploding. The presence of metallic articles, like wire and nails (and of course blasting caps), makes an explosion likely when the flame strikes it. It is essential when destroying dynamite to take it out of its wooden case on account of the nails in the latter.

The cases should be burned separately, as when they become saturated or stained with exuded nitroglycerin they almost always explode when burned; but if no dynamite is mixed up with them there is generally not enough of an explosion to do any great damage.

To set fire to them, soak a little excelsior or waste in gasoline and put one end of a piece of fuse in this, being careful to light the other end first. Have the fuse two or three feet long and the waste or excelsior under the empty dynamite cases. Or you can lay a train of excelsior and light one end with a match and run.

Dynamite cases showing any signs of being stained should never be used for fuel, but should be promptly destroyed in the manner described. If they blow up in the open, with everybody at a safe distance, they will do no harm, but if they blow up in the kitchen stove somebody will probably be badly hurt or killed and the house burned down. Don't take any chances.—E. I. Du Pont de Nemours Powder Co.

NEWS NOTES

Tractor Hauling Road Material—A gasoline tractor for hauling material for road-construction in Minnesota has been employed by Morris, Shepard & Dougherty, contractors, St. Paul, Minn. The tractor was of the Holt caterpillar type, with single steering wheel, and hauling 4-yd. Troy dump wagons. Gravel was taken from various pits, being loaded

Tractor and Dump-Wagons for Distributing Road Material

by a belt conveyor into an elevated bin, from which it was spouted to the wagons, as shown in the accompanying view. The length of haul was from one to four miles. Where the grade permitted the tractor hauled seven loaded wagons, but usually, owing to hills and soft places in the road, it hauled five wagons. The tractor was used also to operate a road grader.

A Plank Floor Carried by Centering for a Concrete Floor—A working floor to permit of teaming over a green concrete floor was a special feature of the construction work on the Orpheum Theater at Detroit, Mich. The contractor found that it would be necessary to use the first floor for handling material, and therefore the centering was designed of sufficient strength to carry a plank floor for teams. The centering was built with 2-in. plank and a large amount of shoring. The concrete floor was completed one night; the next day the plank floor was laid on the concrete, and on the following day teams were driven on this floor to deliver brick, terra-cotta, etc. The contractor was F. H. Goddard, Inc., of Detroit.

Thin Bituminous Coats on Brick and Concrete Pavements—The application of asphalts and tars on concrete and brick pavements has been tried repeatedly in many cities. These experiments have been, in general, a waste of time and money, and have been at best only temporarily successful. While it is undoubtedly true that the application of a bituminous surface on concrete or brick will render these pavements practically noiseless and will tend to retain the dust to some extent, it has been found difficult to build a mat of this type which will not peel or "pick up" in patches. The accumulation of foreign matter on the road has furthermore a destructive influence on the thin bituminous mats used, and bituminous surfaces less than ½ in. in thickness soon wear out.—F. L. Roman, Testing Engineer, Illinois Highway Department, in its Bulletin No. 6 on Dust Prevention.

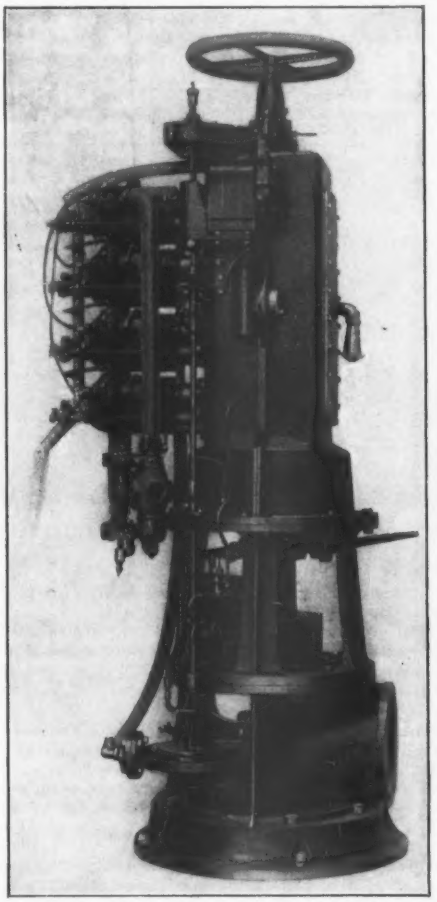
Blasting Frozen Bulk Freight—The Du Pont Powder Co. makes the following recommendations in its house organ for safely expediting the unloading of frozen bulk freight like broken stone, ore, sand and clay:

Use a slow-acting dynamite or farm powder in small quantities, say, one-quarter or one-half of an 1¼x8-in. cartridge to the charge. Start with a very small charge until requirements are gaged.

In placing the charge make a hole with a crowbar in material such as broken stone and with an auger in material such as sand or clay. Do not have any part of the charge of explosive nearer the side or bottom of the car than two or three feet. Tamp the hole when loaded.

Fire the charge preferably with an electric blasting-cap, though a blasting-cap and fuse can be used.

An Interesting Deep-Well Pump Head built by the Hill-Tripp Pump Co., of Anderson, Ind., is shown in the accompanying illustration. This carries a four-cylinder vertical-shaft high-speed oil engine with automatic control for loss of pump suction or drop of load. The company makes a variety of novel centrifugal pump designs in one to six stages and for heads of 10 to 1000 ft. with capacities of 10,000 to 20,000 gal. per min. A particular feature of the impellers is the flaring sides beyond the vanes by which the cross-sectional area of water passage is made gradually to increase.



OIL-ENGINE CENTRIFUGAL-PUMP HEAD

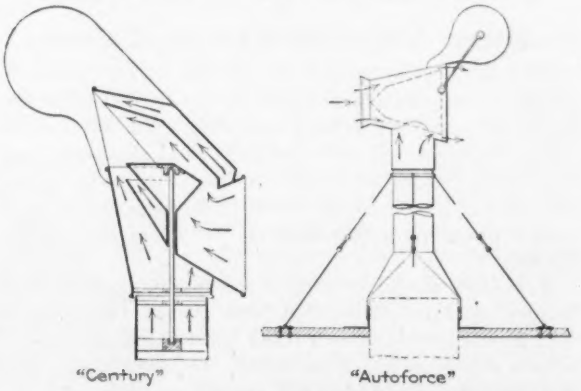
By a reduction of water velocity here friction loss is reduced and pressure increased. An efficiency of 72% is claimed for pumps with this type of impeller. The single-stage pumps also have a patented balancing device consisting of a heavy tapering rib in the middle of the casing chamber opposite the center of the impeller. If the impeller should move slightly to one side the changed deflection of water would cause a restoring unbalance of pressure on one side of the impeller. For the multistage designs one of the familiar "restless" balancing devices is employed—with opposed high- and low-pressure connected chambers and a throttling relief.

Steel Trough Forms for Ribbed Concrete Floors are a specialty of the work of the Concrete Engineering Co., of Omaha, Neb. The troughs, or "steelforms," are steel plates of No. 16 gage, pressed to practically rectangular section, with horizontal flanges which rest on the centering. These flanges are pierced with holes for nails to hold the troughs in place. Along the lower part of each side are ¼-in. holes 6 in. apart for wire ties or hangers for the attachment of expanded metal or wire lath for flat ceilings, which can be applied after the steel troughs are removed. In pressing the plates, depressed ribs are formed across the top of the trough to give stiffness. The troughs are 10 to 20 in. wide on the open side (tapering to 8 to 18 in.) and 6 to 12 in. deep, made in lengths of 3 ft. They are lapped from 1 to 6 in., and the ends are closed by steel plates. Metal supports resting on the flanges of adjacent troughs hold the joist reinforcing bars in proper position. This construction has been used in a number of large buildings.

A Sprinkling System on the Downstream Face of the Roosevelt Dam, the U. S. Reclamation Service dam near Yuma, Ariz., is maintained in order to minimize temperature changes on that face in the summer. The system consists of a perforated pipe passing along the downstream parapet and fed from the same spring which furnishes the water-supply for the camp. The most severe stresses to which the dam is subjected are on the downstream slope when the reservoir is empty. By the increase of temperature on a hot summer day the expansion of the top of the dam increases its length, to accommodate which the curvature must increase, and the top of the dam therefore tips slightly toward the reservoir. This is greatest when the reservoir is empty, and it sets up tensile stresses in the downstream slope which theoretically would crack the masonry. While it is not anticipated that such cracks would do much harm, even if they took place, in order to minimize them the automatic sprinkler is maintained during the summer and the temperature kept nearly uniform. As this water runs down the slope of the dam it has the tendency to collect in places, leaving other places dry, which gives to the face of the dam a mottled appearance.

Transverse Cracks in Concrete Pavements—The occurrence of transverse cracks has been almost as erratic in concrete pavements with joints spaced 50 ft. apart as in those with 100-ft. joints. Past experience now points to the advisability of building concrete pavements with no transverse joints whatever, except that at the end of each day's work the end of the slab might be faced with a piece of wool felt, possibly ⅛ in. thick. This would permit concreting to be resumed the next day, with the least disturbance to the concrete previously placed. It is not to be expected that a slab of concrete some 7 in. thick and several hundred or several thousand feet in length can undergo the process of setting up hard without yielding to the shrinkage stresses necessarily introduced by the rich consistency of the concrete. I believe that the occurrence of transverse cracks cannot be entirely forestalled by any treatment of the subgrade when a concrete of rich consistency is used, and that the occurrence would be almost equally probable and erratic if the concrete pavement were uniformly supported upon a liquid bearing throughout its entire length. It would seem that the difficulties are inherent in the nature of the material, but in all probability they can be reduced by using a leaner concrete, and by the addition of a percentage of inert material, as hydrated lime, for the purpose of reducing the voids in the concrete.—H. E. Bilger, Road Engineer, Illinois State Highway Department, Springfield, Ill.

Automatic Exhausting Ventilators, such as are shown in the accompanying sketch, are meeting with favor for schoolhouse ventilation. In both designs shown a vane adjusts the hood so that wind is forced to blow through and aspirate the air in the ventilation duct. The "Century" design is made by the Century Ventilating Co., 1038 Forest Ave., Bronx, New York City, and a typical installation is seen on the



TWO AUTOMATIC EXHAUSTING VENTILATORS

elevator shaft of the New York High School of Commerce at Broadway and 65th St. The "Autoforce" device is made by the Autoforce Ventilating System of Boston; typical installations are found on the New Lancaster, N. H., High School. The cost runs from about $20 for one of a capacity of 5000 cu.ft. per hr. to $110 for 160,000 cu.ft.

English vs. American Surveying Terms—An English reviewer criticizes an American surveying manual because "poles and staves are 'stadia' (which is very much as though one should call milestones a high road), and the marks are 'monuments.'" Tacheometric surveying, he says, appears unknown—until one finds it described as a "stadia survey."

Editorials

The Bureau of Standards on the Autoclave Test

The autoclave high-pressure steam test of the soundness of portland cement is not so live a matter as it was a year or so ago, but the memory of the persistence—and of the blundering—attending its promotion is sufficiently strong to lend new interest to any unprejudiced expert examination of its merits. Such an examination has recently been completed in the Bureau of Standards, and a paper on the subject will soon be available. Meanwhile, engineers may read the brief abstract by its authors on another page of this issue.

From the last conclusion in that abstract it appears that two years' investigation has convinced the government experts of the futility of the test "as a means of determining the ultimate soundness of concrete." At the same time, all of the other conclusions emphasize the early-noted inconsistencies of results in the autoclave and show definitely that the opposition to the test is based, not so much on what it does not do as on the difficulty of prophesying what it will do. Of course, a test which is inconsistent is no test at all and can have no place in standards.

Conclusion No. 4, without the light of the full report at least, is unconvincing. Does not the application of the adjectives "permanent" and "durable" to the results of a two-year test indicate the common laboratory inability to give proper significance to the relation between test and performance?

The Submarine Torpedo-Boat and Its Results

The sinking of the transatlantic liner "Lusitania" by a German submarine torpedo-boat has stirred public opinion in the United States as no previous event of the war has done. Hitherto, the general public attitude toward the submarine war vessel has been more or less skeptical. The spectacular possibilities of aërial warfare have attracted far more attention. But the sinking of the "Lusitania," even more than the previous destruction of the British cruisers and battleships by an underwater foe, sets the world face to face with the possibilities of this new engine of destructive warfare.

Ever since the first submarine boats were built it has been prophesied that underwater attack would eventually revolutionize naval warfare. Little more than a year ago this view was put forward by one of the leading Admirals of the British Navy. He declared that the submarine boat had already made obsolete the battleship and the huge armored cruiser. A chorus of dissent naturally arose in Great Britain, but the events of the past nine months have again and again demonstrated the soundness of the Admiral's views.

Of course, the argument has been made that the losses of vessels by the Allies through submarine attack have been trifling compared with the total strength of the allied fleets. Against this must be set the fact that the existence of the German submarines has revolutionized naval tactics and prevented the close blockade of German ports by the allied fleets which would otherwise have doubtless been established.

Again, it has been claimed that the subdivision of the hulls of modern warships by water-tight bulkheads has progressed so far that they could not be sunk by torpedo attack as were the vessels in the Russo-Japanese war. But the loss of British cruisers and battleships on the North Sea and elsewhere through the attack of torpedo and mine has shown that the unsinkable ship exists only in the imagination. It is hopeless to expect that vessels can be made which can defy attack from under water by high explosives.

The submarine torpedo-boat represents a marvelous achievement in engineering. As such it is proper to review in this place what is the present status of this terrible engine of destructive warfare and—even more important—what is it likely to become in the future?

As most of our readers doubtless know, the modern submarine torpedo-boat is more properly termed a submersible boat. Under normal conditions it cruises with its deck above water. Using oil fuel and propelled by Diesel oil engines, it has a cruising radius of 2000 to 5000 miles. When the vessel is submerged it is propelled by electric motors supplied with current from storage batteries. Its speed under these conditions is comparatively low. The tactics of submarine operation are substantially as follows:

The cruising submarine, being very low in the water and making no smoke, can generally sight an enemy vessel before it will be itself sighted. The submarine, if it seeks to attack, can then submerge itself until only its periscope is above water, and will then approach as closely to the foe as is prudent without incurring too great risk of discovery and a consequent machine-gun volley at the periscope. It will then disappear entirely beneath the surface, and then comes the greatest limitation to its action. When thus submerged, the crew are in total darkness as to the location of the vessel they seek to attack. They can, of course, navigate their own craft by the compass, but to approach the foe they have to rely solely on their knowledge of its whereabouts prior to submersion and on their assumption as to its course and speed in the elapsed interval.

If the vessel which the submarine seeks to destroy changes its course or its speed, or both, the chances are many that the submarine will miss it. Of course the latter can bring its periscope above the surface to verify the enemy's position before firing its torpedoes, but it does this at the risk of being fired on and put out of action before it can again submerge.

Obviously, the nearer the submarine can approach a vessel before total submergence, the better its chances of success. In clear weather the submarine can scarcely

operate against naval vessels. Its opportunity for approach must be found under cover of fog or storm or darkness. Unarmed merchant vessels, however, it can approach with impunity and effect their destruction as it pleases, subject, however, to the risk that the merchant vessel may carry machine guns for protection against such attacks.

It is not at all a far stretch of the imagination, however, to foresee an invention that would render the present submarine torpedo-boat far more effective as a weapon of destruction. It needs only the development of some means by which the crew, when their boat is submerged, can determine the location of the enemy's vessel which they seek to strike. We cannot say, of course, whether such an invention is possible or probable; but the marvelous developments in wireless telegraphy and other applications of electricity and magnetism would indicate that by some delicate instrument it may be possible for those in an underwater boat to determine the direction in which a steel-hulled vessel lay.

If such an addition to the submarine boat's equipment were made, there seems no escape from the conclusion that a fleet of such vessels would have the navies of the world at its mercy. Against such a hidden foe, able to navigate the depths at its leisure and direct explosives against the vulnerable underwater hull of a vessel, neither guns nor armor nor bravery would avail anything. It is possible to send aëroplanes aloft to attack enemies in the skies, but it is hardly to be hoped for that submarine boats could successfully find and attack other submarine boats in the ocean depths.

There is no reason to doubt, moreover, that the submarine boat will be further perfected, if not by such an invention as we have suggested, then by others. Its cruising radius will be increased. It may be given greater speed under water and the weapons it uses may be improved. Modern civilization may indeed recoil at the deeds of this modern Frankenstein, but so long as civilization can find no way to limit the use of the terrible forces which are now available for destruction, it is the part of wisdom to face the facts squarely.

And the first of these facts is that the submarine boat is the most powerful weapon for sea defense that has ever been developed. Even a small and weak nation can protect itself from attack from overseas by the possession of a fleet of submarine boats, of which a dozen or a score can be built at less cost than a single battleship.

There has been much speculation during the past six months as to the possibilities at some future time of an armed expedition crossing the ocean to attack the United States. Without taking into account the force which our other naval vessels could oppose to any such expedition, the fate of the "Lusitania" is an object lesson as to the defense which American submarine vessels could make against any such attack. The United States has at present nearly 60 submarine torpedo boats and there can be little doubt that the events of the past few months will cause this number to be doubled or tripled at an early day.

Further than this, when the submarine boat is further developed, in the way we have suggested, an end will come to the era of sea power, which has been such an important factor in the past. The costly rivalry to possess the strongest navy will be ended, for it will be useless to build battleships and cruisers that are helpless against underwater attack. No nation can successfully attack another by overseas invasion, for the ships would be sunk as fast as they came.

Years ago a philosopher said that civilization must contend for its life with the forces that modern science has created. If the submarine can put an end to the burden of great navies its ultimate influence may be beneficial, but the possibilities must also be faced of the use of this destructive weapon against the commerce of the world. Little more than a century ago the chiefs of the Barbary coast levied tribute on the merchant ships of all nations and sold captured crews and passengers into slavery. The possibilities of a fleet of submarine boats in possession of an outlaw among the nations are not a pleasant subject for speculation. One of the great questions to be settled in the outcome of the great world conflict now raging is how the use of these and other great destructive agencies can be controlled.

The Jitney Bus vs. the Trolley Car

Three or four years ago those interested in street railways in England awoke to the fact that the motor omnibus had become a very serious competitor of the electric railway car. In the United States the motor omnibus has come into extensive use in country districts, especially in regions where good roads were available, but it has as yet cut no important figure in city transportation. In the short space of five months, however, the "jitney"-bus movement, spreading from the Pacific to the Atlantic, has for the first time aroused the street-railway fraternity and the public to the possibilities of this new means of city-passenger transportation.

We publish elsewhere in this issue a paper on the jitney bus read last month at the meeting of the Iowa Street & Interurban Railway Association. The author of the paper, C. I. Palm, concludes that the operating cost of the jitney bus is too great for it to long continue a serious competitor of the street railway, and that with proper regulation by municipal authorities the jitney craze will subside as quickly as it came. He freely admits, however, that its existence is a very serious threat to the street-railway business and that the motor omnibus with large passenger capacity has probably a future in city-transportation service.

Mr. Palm's figures on the cost of operation of jitney busses are worth careful study, for, admittedly, the question whether the jitney has come to stay depends more than anything else on whether those running them can make the business profitable. Mr. Palm estimates that the cost of running a Ford car 100 miles a day, not including the wages of the driver and depreciation on the car, is 3.7c. per mile. This compares very well with the figures given in our issue of Apr. 22, 1915, p. 769, as the result of experience with three cars used on road-supervision work in the South. The average expense for the three cars run there was 3.94c. per mile run.

Included in Mr. Palm's figures is an item of 1c. per mile for repairs, storage, supplies, etc. This is a charge of $1 a day if the car averages 100 miles per day. It is doubtful if the average jitney owner who keeps his car in an unused stable or in a cheap portable garage and makes nearly all the repairs himself spends anything like $1 a day for these items.

It is in computing the earning capacity, however, that Mr. Palm's demonstration that the jitney owner is up against a losing game seems to us most defective. He computes that for a Ford car seating four passengers the operating cost will equal 1.78c. per passenger seat per mile. But numerous operators of jitney busses in the West have had Ford and other pleasure cars transformed by the substitution of an omnibus body so as to carry eight or ten passengers. In any city having easy street grades such a transformation is feasible without interfering much with the speed of the car. With a car carrying eight passengers, the cost per passenger seat mile becomes only 0.89c.

Mr. Palm assumes an average passenger trip of 2½ miles. At this rate, when the bus can average a half-dozen passengers, running 15 miles per hour, its earnings will amount to $1.80 per hour, or a profit of 75c. an hour above the operating cost of $1.05 per hour assumed by Mr. Palm, in which operating cost there was included wages of $3 per day for the driver. This at once transforms the jitney-bus business into something exceedingly attractive to the average mechanic or street-railway man or clerk who has saved enough money to buy a second-hand motor car and put an omnibus body upon it. It is, moreover, not to be supposed that the jitney-bus drivers are going to handle the unprofitable long-distance traffic into the remote suburbs. They will let the street railways handle that. The jitney drivers choose their route where the traffic is heaviest and will run no further away from the centers than they can obtain paying passengers.

In most cities there are routes which are not now served by street railways where profitable traffic can be obtained for motor-bus lines. On Broad St., in Philadelphia, for example, a large number of jitney busses have been started within the past few weeks. At the rush hour in the evening the City Hall plaza is said to be crowded with passengers waiting for a place on the busses.

It must be remembered that to hundreds of thousands of people, automobile riding still has the attraction of novelty. Even though a jitney bus is not a limousine, it is apt to be more comfortable for its passengers, in pleasant weather at least, than the average street-railway car at the rush hour.

In many cities, as Mr. Palm points out, the jitney business has been overdone. It is doubtless for this reason that the number of jitneys in Los Angeles is said to have dropped from 1200 to 700. The jitneys are competing with one another at the same time that they are competing with the street railways. It will be agreed also that the motor omnibus and not the second-hand pleasure car is likely to be the serious competitor of the electric street railway in the future.

We published a year or so ago figures of cost of operation of motor omnibuses in London, showing a total operating cost, including wages of driver and conductor, of only 14 to 17c. per mile. While these costs are influenced by the low rate of wages prevalent in England, it is to be remembered on the other hand that the cost of gasoline is much higher there than in the United States. The City of York, England, as we reported in a recent issue, has established a line of electric omnibuses, which are so simple to manage that the driver is able to collect fares also, the entrance door being beside him at the front. Thus the wages of the conductor are saved and operating expenses are kept low.

Mr. Palm expresses the opinion that the motor bus is in the future to be the *de luxe* system of public street transportation, carrying passengers who are willing to pay a higher fare than 5c. for the sake of greater comfort and speed.

The figures above quoted, however, show that if the motor bus can secure passengers enough, it need charge no more than 5c. to make its operation profitable. The motor-bus operators will, of course, do whatever yields them the most profit. If they can do a better business on 5c. fares than on 10c., they will not keep up their price because of consideration for the street-railway companies.

That the motor-omnibus business should be subjected to municipal regulation will be generally agreed. Bonds to insure responsibility in case of accident are already required in a number of cities, and operators should be required to have some certificate of competency. The street-railway companies in some cities are seeking to add to this a heavy car-license fee, as a means of discouraging these competitors; but is doubtful if measures of this sort will attain much success. The new mode of transportation is popular with the public, and any attempt to throttle it by unfair measures is likely to react against the street-railway companies.

The Failure of the Great German Crane of the Panama Canal

Through the courtesy of Gen. George W. Goethals, we are able to present in this issue of ENGINEERING NEWS the first authoritative and complete account of the failure under test load of one of the floating cranes purchased for the Panama Canal, prepared by Civil Engineer Cooke, U. S. N., a member of the Panama Canal staff. Not since the fall of the Quebec bridge has an accident occurred of such interest and importance to the structural engineer.

The crane that failed, while as large as any in existence, was of approximately the same size and capacity as a number of others that have been in use for several years. The German builders of this crane have had probably the widest experience of any firm in the world in the construction of very large floating cranes. The crane is a type of engineering structure in which it is possible to compute the stresses with greater accuracy and certainty than in almost any other structure which the engineer builds. A railway or highway bridge, in comparison, is subjected to impact forces from the passage of loads at high speed, concerning which there is admittedly a considerable factor of uncertainty.

Again, what stronger assurance could one have of accurate work in design and sound work in construction than the knowledge that the work had been executed by a German firm of such large experience and international reputation? Americans have at times criticized German engineering for the great attention paid to theoretical studies and investigations. Certainly, then, one would feel every assurance that structural work executed by a first-class German firm would have a sufficient margin of strength in all members where the stresses were capable of exact computation.

There was as little reason, therefore, to doubt the ability of the Panama crane to handle safely its test load as

there was to distrust the strength of the Manhattan bridge or the steel framework of the Woolworth Building. Yet, in the face of all these facts, when the first German crane for the Panama canal, the "Ajax," was set to lifting the contract test load, 20% in excess of its rated capacity at extreme reach and when the heavy weight of 134 tons suspended 82 ft. beyond the fender was raised 6 ft. off the ground, the overhanging jib slowly crumpled and dropped its load to the ground, and then the jib itself fell on the wharf a shapeless, twisted wreck.

It is doubtless because no person was killed or injured in the wreck that details of the accident did not get into the newspapers, so that instead of the story being told piecemeal to the engineering profession it can now be presented complete, after full investigation into the causes which brought about the accident.

The structural engineer will need only a brief examination of the photographs and drawings accompanying Civil Engineer Cooke's account of the accident to perceive at once the close similarity between the causes of the Panama crane failure and the causes of the Quebec bridge failure. Summed up in few words, the verdict in each case is lack of stiffness in large compression members.

It was, of course, a very different type of compression member that failed at Panama from the huge box that formed the compression chord of the Quebec bridge, but the initial failure which occurred in strut 29, like the initial failure in chord *A9L* of the Quebec bridge, was due to inherent lack of stiffness. This at least is the conclusion of the engineers who have given long and painstaking study to the causes of the failure.

It is fair to say at this point that the German builders of the crane do not share the opinion that there was lack of stiffness in compression member 29. Their claim is that in the previous use of the "Ajax" to hoist the wreck of a drill-barge, the full load was for a time carried on only one of the two lifting hooks of the crane,* and this put nearly the whole load on one truss of the jib. This, it is thought, subjected parts of the jib to stresses greater than they were designed for, and it is suggested that the overstressing on that occasion started the failure which was completed when the test load was applied.

It seems to us, however, that the shearing of the rivets of the lattice bars that held together and braced the two parts of strut 29 is sufficient indication that these connections were not strong enough to perform their duty. Many of the readers of ENGINEERING NEWS will recall the extended discussion on the stresses in lattice bars and lattice-bar connections which was carried on in this journal following the failure of the Quebec bridge. It is very likely the case that German structural designers did not benefit by these extended studies concerning the stresses in compression members, as did their American brethren at that time.

Others also may have failed to draw full instruction from the Quebec collapse. The builders of the crane inform us that the latticing of this member was put in at the request of the Panama Canal and that the builder's intention was to provide transverse battens or tieplates at intervals, instead of latticing. If this be true, and if the Panama Canal, after asking that lattice be substituted for battens, approved the detail drawings of the lattice, and therewith approved the one-rivet lattice connection, then the Canal engineers will bear a part of the blame for the latticing that proved to be deficient and allowed the heel-strut to buckle.

And now, turning from the general aspects of the case to the definite calculable elements of the accident, we find that a most important deduction can be made in regard to proper provision of transverse strength in columns. After the Quebec failure it became good practice to specify for very large compression members, that they must be able to carry a transverse shear equal to a certain fraction of their longitudinal load—usually 2 to 3%. In ordinary bridge specifications it is not necessary to deal with this feature. Still, we find in the specifications of the New York Central Lines—one of the most thorough among railway-bridge specifications—a clause on latticing of columns, that calls for a transverse strength substantially equal to 2½% of the longitudinal load. U. S. Government specifications, we believe, have in some cases fixed the ratio at 2%. How do these figures bear on the failure of the crane "Ajax"?

With a sectional area of 48.1 sq.in., the heel-strut of the "Ajax" was designed to bear a maximum compression of about 577,000 lb. The two planes of lattice of the strut, each with one ¾-in. rivet, would ordinarily be credited with a safe working strength of 9000 lb.; in other words, if the strut had to resist a transverse shear of 9000 lb., or 1.6% of the maximum longitudinal load, the latticing would have been called adequate. Yet this lattice failed. Plainly, it would not have been satisfactory if the designers had provided lateral strength for 2% of the longitudinal load, or even 2½%, as the New York Central requires. For even in the latter case the lattice would have had a margin of strength of only 56% above its failure point, whereas we usually provide a safety-margin of at least 150% in structural parts.

To give a margin of 150% (equivalent to a factor-of-safety of 2½) the latticing would have had to be designed for a transverse force of $2\frac{1}{2} \times 1.6\%$, or 4% of the longitudinal load on the strut.* This shows that our highest specifications for transverse strength of columns are at least one-third too low.

Doubtless, some regard must be had to the fact that this heel-strut of the Panama crane was a very unusual column. It was not braced at its upper end, except by a pin attachment to another strut and to the main pulling link. Conditions favored its lateral deflection, and any deflection would produce a transverse component, stressing the lattice and its rivets correspondingly. Ordinarily, struts in structural work are not so unfavorably arranged.

While this fact may modify the conclusion above indicated, namely, that a transverse-strength provision of at least 4% seems to be necessary, yet the amount of such modification can hardly be estimated. The safe course is to make latticing strong enough for 4% of the column load.

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*Photographs showing the drill barge hanging on one hook make this clear. In addition, a report on the accident, by the Panama Canal engineers to the English underwriters, rendered in January, describes in detail the rigging and procedure for lifting the barge, and this report tells of lifting the wreck out of the mud with one hook and later attaching the other hook, etc. The overloading thus seems to be established; the matter of overstressing is obviously open to discussion.

*Still higher percentages are found if the probable ultimate shearing strength of the rivets is taken into account. The lattice-rivets were stressed exactly to their ultimate strength—they held for a time, but then failed at a slight disturbance. Their strength, at 35,000 lb. per sq.in.—a low estimate—was 31,000 lb. (for the two rivets). The total longitudinal stress in the strut was about 577,000 lb. The transverse shear, therefore, was nearly 5½% of the load.

Letters to the Editor

Design of Reinforced-Concrete Standpipes

Sir—In ENGINEERING NEWS, Apr. 29, 1915, there is a condensed history of the Attleboro standpipe and also a letter from L. J. Mensch, of Chicago, commenting upon its design and construction. I am glad that Mr. Mensch is directing attention to the fact that if many of these concrete standpipes have not been water-tight the leaking is the inevitable consequence of the design. I became interested in the construction of reinforced-concrete reservoirs at about the time the Attleboro standpipe was being erected. While I had no connection with the building of it, I was in possession of considerable information regarding it and was familiar with the details of the design. From my own study of the question at that time I arrived at conclusions conforming to those set forth by Mr. Mensch.

In 1910 I submitted to the annual meeting of the American Gas Institute a discussion on tanks for gas holders, in which concrete tanks are discussed. When I wrote that discussion the Attleboro standpipe was one of the reinforced-concrete reservoirs in my mind. The paper contains a diagram illustrating my conception of the deformation of a masonry tank; although Mr. Mensch's illustration and my own are pictorially different, it is evident that our analyses are identical.

While the truths presented by Mr. Mensch should be disseminated, I am a little skeptical as to their reception, for I have had some contact with engineers promoting reinforced-concrete construction, and I know that they frequently meet conditions in which they can only spell out economy or advantage for that type of construction by resorting to the fallacies condemned by Mr. Mensch.

I have in my possession several designs for large reservoirs proposed by some of the prominent engineers in this country engaged in reinforced-concrete construction. It is appalling to think that they actually urged the investment of large sums of money in structures designed in defiance of the fundamental principles of mechanics. If tanks had ever been built from these designs, they most certainly would have deteriorated rapidly. We cannot defy the laws of creation and get away with it.

To avoid misunderstanding, let me say in closing that I believe in reinforced-concrete construction—where it belongs.

HERBERT W. ALRICH,
Consolidated Gas Co.
4 Irving Pl., New York City.

Sir—The writer, who is both the designer and builder of the standpipes at Attleboro, Mass., and Westerly, R. I., has read very carefully in your issue of Apr. 29 the articles on the Attleboro standpipe.

In the history there are two slight points which the writer wishes to comment upon. First, at the end of the fourth paragraph from the beginning of the article you imply that there was a movement of the steel in the forms when the concrete was placed. Permit me to say that the reinforcing bars were 1½ in. diameter; a single ring weighed over 900 lb., and was supported at 15 points around the circumference in a very rigid manner. Before the forms were in place it was possible to walk up the bars like on the rungs of a ladder half way between the supports, and there was no appreciable movement.

In the middle of the next paragraph you state that mortar was forced into cavities between and around the clips by throwing it a distance of 4 to 5 ft. The repairs were done from such a stage as is shown in your article, Figs. 2 and 3, and so you will see it was impossible for the men to get to any distance to throw the material. It is the writer's recollection that it was applied in the usual method of plastering work.

It is the writer's opinion from observation of the tank, after it had been put into service, that the concrete was forced off over large areas outside of the reinforcing hoops by water which, after leaking out of the tank, reached the outer line of hoops and was deposited in minute pockets close to the steel caused by the unequal movement between the steel and the concrete. This water was probably sufficiently near the surface to be frozen in cold weather, thus expanding the pocket. More water followed and froze, and by the repetition of the process a line of cleavage was made on the line of the hoops, where the concrete area was reduced by nearly half. As concrete is brittle, a crack once started will run a considerable distance; as when the iceman makes a slight crack in a block of ice with his chisel the crack immediately runs through the whole cake, splitting it. In this way a large area of concrete would be split off at some distance from the source of leakage. As the hoops were coupled mechanically to develop more than their working stress, irrespective of bond with concrete, the tank was in no way structurally weakened by the loss of this finished surface, although its appearance was impaired. It would then be like a wooden tank with exposed hoops outside.

After the tank was repaired by the waterproof membranous lining, the outside could have readily been repaired with cement, with assurance that it would have remained in good order, with very much less expense than has been put into the brick covering.

The working stress in the reinforcing hoops of the Attleboro standpipe was taken as 14,000 lb. per sq.in. There is, of course, some variation in this stress, due to a fixed spacing of bars for a height of about 3½ ft. vertically. At the very bottom the actual stress is 13,800 lb. With this stress the lengthening of a hoop in the whole circumference is 0.916 in., and using this elongation and a modulus of elasticity of concrete of 2,000,000 lb. would give a stress in the concrete of 1115 lb. per sq.in. Prof. F. E. Turneaure states that concrete cracks when the stress in the steel is 5000 lb. or less. Using 5000-lb. stress in the hoops of this standpipe would give an elongation of 0.327 in. The difference between this elongation and that at 14,000 lb., which is the actual

stress when the tank is full, is 0.589 in., or we may expect the concrete to be stretched more than $\frac{9}{16}$-in. in the whole circumference beyond the point at which it will crack; and as these cracks probably are concentrated at intervals rather than being uniformly distributed, we may expect in the aggregate cracks amounting to $\frac{9}{16}$ in., and some of these will surely be large enough to permit leaks.

At Westerly a working stress at the bottom of the tank of 6000 lb. was used, which increased 1000 lb. for each foot in height until a stress of 13,000 lb. was reached, which was used for the rest of the height. This lower stress reduced the amount of leakage at the bottom of the walls, where it has been greatest in the Attleboro tank, and the leakage as a whole was very much less than in the other. As near as the writer was able to estimate, it did not exceed two quarts in 24 hr., and there were fewer leaks in joints between each day's work than there were through the middle.

In observations on the Attleboro standpipe it appeared that when the tank was filled full and kept practically full the leaks which appeared at the bottom gradually silted up and became almost negligible, but when there was very large variation in the head of water the cracks which had silted up were broken down by the elasticity of the steel compressing them as the pressure of water fell. During the summer time, in the early days of the tank, the variation in head was as great as 40 ft. per day. Therefore the cracks had no opportunity to silt up and remain tight.

You state ". . . that ten years ago engineers and contractors did not consider it worth while to bestow that scrupulous study on materials, mixing and placing, that it is now realized can alone produce dense concrete, watertight under high heads." Permit me to call your attention to a paper entitled "Reinforced-Concrete Standpipe," presented to the American Society of Civil Engineers, Paper No. 1213, appearing in the "Transactions" of Vol. 74, p. 375, year 1911, written by W. W. Clifford, describing the methods of analyzing materials in connection with the Westerly standpipe. Mr. Clifford was timekeeper and engineer of the job under the direction of the writer, and he followed the writer's instructions and used the method and the same apparatus that had been previously used on the Attleboro standpoint. I think you will agree that the methods used in this were painstaking and thorough. In the light of eleven years of further experience I do not know how to use any more care than was then used.

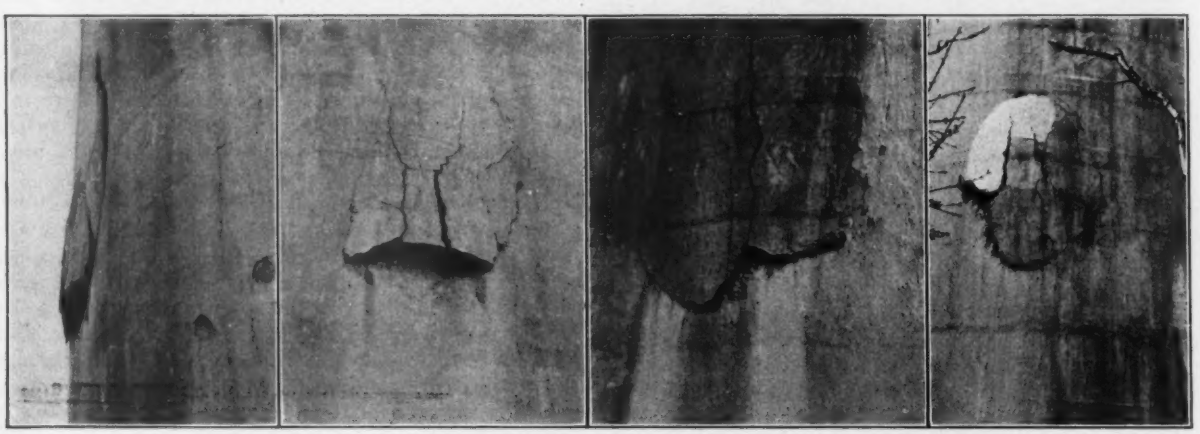
The radical change in the design of big standpipes which the writer would recommend is to recognize the limitation of the material, namely, that concrete is rigid and relatively inelastic and therefore ought not to be expected to stretch and resist cracking and leaking. In designing a future standpipe, the writer would consider merely structural stability as in the case of a building, and use no greater care in the placing of reinforcement and concrete, and without reference to its waterproofness. Then put an elastic membranous seal, such as you describe was used by F. W. Bird, on the inside as a waterproof lining, and thus a tank could be guaranteed water-tight. The saving in the cost of labor and materials, if stability without waterproofness is the object, would save enough money to more than pay for the membranous lining.

LEONARD C. WASON,
President, Aberthaw Construction Co.

Boston, Mass., May 6, 1915.

[The theory of eliminating cracks in the concrete walls of standpipes—and consequently the leakage through these walls—by restricting the steel stresses to very low values was fully and favorably discussed in ENGINEERING NEWS four years ago in commenting upon a paper entitled "A New Theory for the Design of Reinforced-Concrete Reservoirs," presented to the Boston Society of Civil Engineers by H. B. Andrews. Any reference to that theory was avoided in our recent editorial discussion because two of the latest high standpipes designed according to that theory show leakage in more or less aggravated form (see accompanying views of the Lexington standpipe). It was precisely for this reason that we advocated a comparative analysis of methods and results in existing standpipes, so that, if possible, the proper relation between theory and performance might be studied.

As Mr. Wason points out and as Mr. Clifford's paper shows, in the Attleboro standpipe the greatest care was taken to insure the proper gradation of aggregate, but we still insist that in 1905 the testing of aggregate was not nearly so fine an art as it is today, when gradation is only one of the many elaborate tests some engineers recommend and practice. Whether these added precautions guarantee a better concrete is perhaps another matter.—EDITOR.]



PRESENT APPEARANCE OF REINFORCED-CONCRETE STANDPIPE, LEXINGTON, MASS.

(Views show four "boils" from 12 to 15 ft. up on all four sides of standpipe; many other slight leaks; standpipe completed two years ago, has never been full; now has about 80-ft. head; no interior lining; inside diameter 30 ft., height 104 ft. 6 in., capacity 550,000 gal.)

High-Pressure Steam Test for Portland Cement*

By R. J. Wig† and H. A. Davis†

The use of high-pressure steam as a means of determining the soundness and cementing quality of portland cement was advocated as early as 1880 by both Michaelis and Erdmenger. It has been recommended and used by others from time to time until recently, when a form of this test was advocated and introduced into specifications for the purchase of portland cement, as the Force Autoclave Test.‡ Many brands of American cement would not regularly meet the requirements of this test, and its recent introduction led to a controversy between cement manufacturers and consumers. The manufacturers of portland cement contended that this test was an abnormal one and that the behavior of a cement when so exposed was no criterion of its ultimate behavior as normally used in concrete. Furthermore, it was suggested that this test be introduced into the United States Government specifications for portland cement and it was for these reasons that an investigation of the value of the high-pressure steam test was undertaken.

This investigation included a study of the physical properties of cements that were sound after exposure in high-pressure steam and cements that failed to meet the requirements of this test. Tests were made of the tensile and compressive strength in mortars, compressive strength in concretes and the linear change of neat-cement prisms stored in air, in water, and treated in high-pressure steam.

The qualitative high-pressure steam test consisted of subjecting a pat of neat cement, after storing for the first 24 hr. in a damp closet, to steam at a pressure of 300 lb. per sq.in. for one hour in a steam-tight boiler, the total time in the boiler being three hours. A cement was said to have passed this test if the pat was hard and sound when examined after this treatment. The quantitative high-pressure steam test consisted of molding six neat-cement briquettes, storing them 24 hr. in a damp closet, then treating three of them in steam at 300 lb. pressure for one hour in a steam-tight boiler, the total time in the boiler being three hours. The six briquettes were then broken in a cement-testing machine, and a cement was said to pass the high-pressure steam test if the treated briquettes showed higher tensile strength than the untreated.

Eighteen 2-bbl. samples of cement, composed of seven different brands, were obtained for testing in concrete. The cement of some of these samples passed the high-pressure steam test and the cement of other samples did not. The concretes prepared from these cements have been normally exposed to the atmosphere for two years or more and tested for compressive strength from time to time. Sufficient specimens were prepared to continue the tests for ten years.

The following is a brief summary of the results of this investigation:

The general soundness tests show that some cements mixed neat, which are sound according to the standard atmospheric steam test, but unsound in high-pressure steam exhibit signs of unsoundness when stored under normal conditions in dry air. This unsoundness may require nine months or more to develop in neat-cement specimens.

The strength tests of mortars and concretes fail to show any difference in the cementing quality of cements that pass the high-pressure steam test and those that fail to meet the requirements of this test.

There is but small difference, if any, in the linear change of prisms made of cement that passes the high-pressure steam test and prisms made of cement that fails to meet the requirements of this test whether stored in air or water.

Of a total number of 51 brands of portland cement tested, 70% passed the high-pressure steam test.

Cements normally unsound in the atmospheric or high-pressure steam tests will generally be found to be more sound if the coarser particles are removed. While fineness is not essential to soundness, it appears usually to be the coarse particles of a normally unsound cement that cause the expansive action resulting in cracking and disintegration of the hardened cement in the accelerated tests of soundness.

A cement originally unsound in the high-pressure steam test will usually be found sound if exposed to this test after aging from two to six months.

The following conclusions are drawn from the results obtained:

1. The high-pressure steam test should be made on all cements that are to be incorporated in cement mortar or concrete products that are to be cured in steam at pressures above atmospheric.

2. The high-pressure steam test may be of value as forecasting the behavior of neat cement or a very rich mortar when exposed under normal conditions in dry air, but it does not forecast the behavior of cements in concrete as normally exposed.

3. Cement passing the high-pressure steam test is not superior in cementing quality, as determined from the compressive strength of concretes, to cement that fails in this test.

4. Cement passing the high-pressure steam test does not make more permanent or durable concrete than cement which meets the requirements of the standard specification, but fails in this test.

5. Cement failing to pass the standard-specification atmospheric steam test, but meeting the other requirements of the standard specifications, shows in some instances a normal strength in concrete.

6. For practical work under normal conditions of construction, the results of this investigation fail to show that the high-pressure steam test is of value as a means of determining the ultimate soundness of concrete.

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*Abstract of Technologic Paper No. 47, U. S. Bureau of Standards, entitled "On the Value of the High Pressure Steam Test of Portland Cement."

†U. S. Bureau of Standards, Washington, D. C.

‡See references in "Engineering News," as follows: June 13, 1912, pp. 1111, 1136; Aug. 8, 1912, p. 263; Aug. 29, 1912, p. 408; July 3, 1913, p. 40; July 10, 1913, p. 82; Aug. 7, 1913, p. 272; Feb. 19, 1914, p. 426; Mar. 5, 1914, p. 528; July 9, 1914, pp. 100, 104.

State Ownership of Railway Freight Terminals in Boston and vicinity is proposed by a bill now before the Massachusetts Legislature. The bill is said to have the backing of Mayor Curley, of Boston, and provides that a state-owned company may acquire and operate all freight terminals in a zone within 15 miles of the capitol. The business would be handled by a board of directors named in the bill, the vacancies to be filled by the governor. All property leased would be rented on the basis of 4½% of appraised valuation, and taxes would be paid as if privately owned. A bond issue for $5,000,000 is authorized to finance the company, the bonds to be guaranteed by the state.

Paving for Piers, Warehouses and Garages

An important point for consideration in the design of shipping piers, warehouses, freight houses, etc., is the best paving material to be used for the driveways and floors. In the former case, there is heavy team traffic to be considered, while in the latter case the traffic consists largely of hand trucks, but the service may be very severe upon the flooring. A similar consideration, but under rather different conditions, occurs in connection with the floors of commercial garages, wagon sheds, fire-engine stations, etc., where there is frequent movement of heavy motor vehicles or horse-drawn vehicles.

We have made inquiry as to practice and experience in paving for these particular conditions, and the information obtained is given below:

Paving for Piers and Warehouses

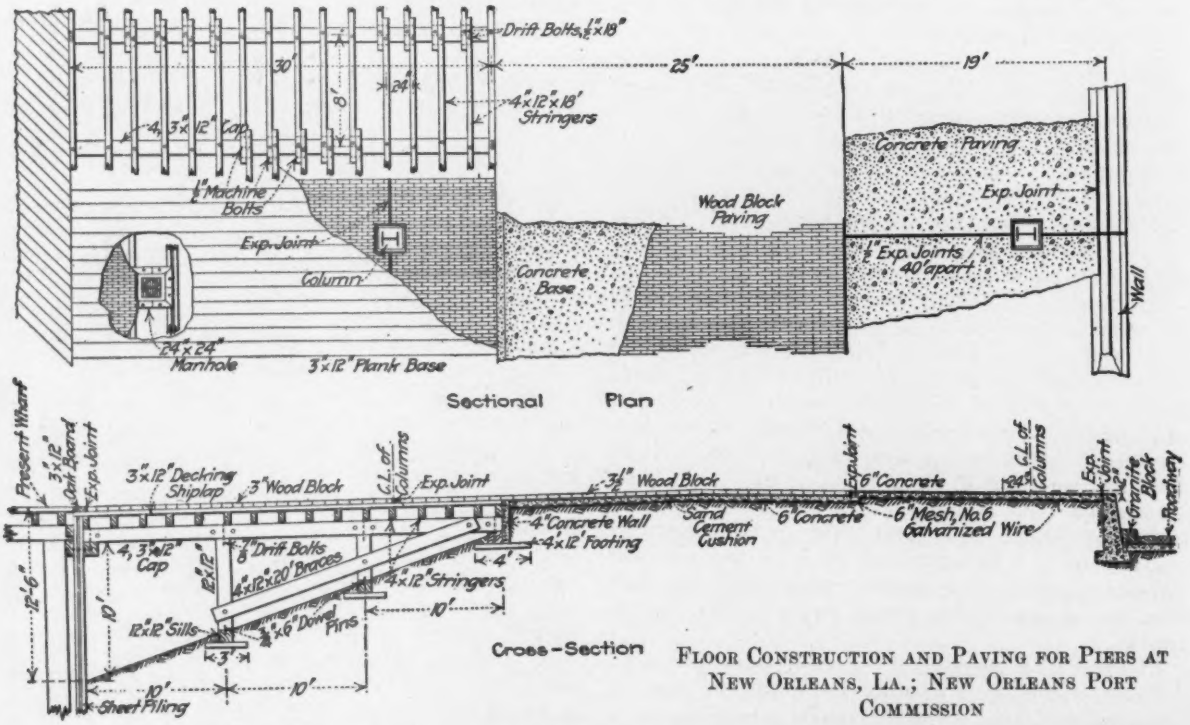
Chicago, Ill.—The new municipal pier, now under construction, will have a central roadway, 80 ft. wide, paved with granite blocks. There will be four tracks in the roadway, with rails laid flush with the paving so that wagons can back up readily to the freight floors or platforms. On each side of the roadway will be a freight floor 100 ft. wide, extending to the edge of the pier. These floors are to be for trucking, and will be paved with creosoted wood blocks on a concrete base. Above the freight floors will be the passenger deck floors, 67 ft. wide, which will be of concrete. Over these will be a concrete roof on steel trusses. E. C. Shankland is Chairman and Chief Engineer of the Harbor and Subway Commission.

Baltimore, Md.—Where there is sufficient travel to keep asphalt in good condition, this material is considered preferable to other pavement in the driveways of docks, sheds and warehouses; the foundation to be 4 to 6 in. concrete, depending upon the character of the subfoundation. If there is not sufficient traffic to keep the asphalt more or less rolled, vitrified brick paving would be preferred. If Belgian blocks are used, it is hard to keep the paving clean and well drained unless given a good crown.

As to the floors of dock sheds and warehouses, concrete with a patented topping has given good results. The stevedores, however, object to it, as it is hard on their feet in trucking. In some cases where heavy steel members, rails, etc., are unloaded on the floor, the surface becomes chipped. But this is restricted to particular parts of the floor where materials of this kind might be unloaded, and as the paving can be repaired readily at moderate cost, this objection is not sufficient to condemn the material. On the upper floors of warehouses, the best results have been obtained with flooring of No. 1 maple. This is somewhat expensive, but it gives a hard, smooth surface, is easy for trucking, and is very durable. O. F. Lackey is Harbor Engineer.

Philadelphia, Penn.—A number of different kinds of paving have been tried on piers in the last few years, but no type has been found entirely satisfactory under all conditions. At the entrance to piers, where the traffic is highly concentrated, and upon the inclines leading from the street level up to the level of the main decks of wharves, granite blocks are used for the double purpose of resisting wear and affording a better foothold for horses. This paving, while exceedingly durable, is not suitable for the main deck paving of the piers, where much hand-trucking is done, on account of its roughness and high tractive resistance to the small wheels of the trucks.

For general use on the main decks, on such portions as are in constant use, creosoted wood-block paving gives excellent results and is considered the best (although about the most expensive) type of pavement yet tried. Steady service appears to be required, however, to make



Floor Construction and Paving for Piers at New Orleans, La.; New Orleans Port Commission

this kind of paving satisfactory, as in all areas not subjected to heavy traffic by teams or trucks the blocks are found to dry out, shrink and become loose, and the pavement deteriorates seriously within 12 or 18 months from the time of laying. As all of the wood-block pavements are laid upon immovable concrete foundations, it is held that these troubles cannot be attributed to foundation faults.

For the less used areas on the piers, two types of cheaper paving are being tried and so far they have proved satisfactory. One of these is a compressed concrete made by rolling the stone and grouting it, instead of mixing and laying it in the ordinary manner; the other is the bitulithic pavement, as used in road-building practice. These types cost considerably less than half as much as the wood-block paving, and if they continue to give satisfactory service they will be adopted as standards for use where the pavement is only subjected to hand-trucking and occasional teaming. John Meigs is Director of the Department of Wharves, Docks and Ferries.

NEW ORLEANS, LA.—In regard to experience with interior paving for heavy service, it is stated that the conditions prevailing along the harbor front at New Orleans are unlike those of any other in the United States. The Mississippi River within the limits of this port has a maximum depth in mid-channel ranging from 100 to 200 ft., and at many points the slope of the bank is rather steep. There is no rock subfoundation and the banks (of clay and fine sand) are more or less unstable. On several occasions during the past ten years, there have been caves which carried away portions of the wharves. As a pavement is not better than its foundation, the problem of permanent pavement along the harbor front is rather difficult and uncertain, particularly within sheds which are built partly on pile and timber structures and partly on the adjacent levee. Immediately behind the sheds, there are paved roadways, which in general hold up very well.

Most of the sheds are floored with untreated 3-in. pine boards, nailed to untreated pine stringers. Where these boards are very near the ground, decay is rapid, and the engineers are considering the replacing of these floors with more permanent construction. In one shed there is a creosoted wood-block roadway which has been subjected to heavy service for five years and which has been very satisfactory, except where settlement has caused the 8-in. concrete base to crack.

A contract has been let recently for a floor in a shed where the traffic will be very heavy, and the accompanying cut shows the construction which has been adopted. The outside driveway is of stone blocks on concrete, with a row of large granite blocks as a curb or wheel guard along the side of the shed. The interior will have a 6-in. concrete floor for 19 ft. along the driveway side, and then a wood-block floor for 55 ft. Part of this wood floor is on a 6-in. concrete base (with sand cushion course) on solid fill, while the remainder is on a course of 3x12-in. ship-lap planking carried by 4x12-in. joists on timber trestling over the slope of the fill.

The concrete paving and base have galvanized-wire netting of 6-in. mesh embedded near the bottom of the slab. Expansion joints (½ in. wide) are provided in both directions across the floor, as shown, and around all column bases and manholes. Samuel Young is Engineer of the Board of Port Commissioners.

SAN PEDRO, CALIF.—This is the port or harbor of Los Angeles, and all the construction work is of such comparatively recent date that no experience has been obtained with paving materials other than pine and cedar wharf planking. The latter is being placed in all the transit sheds for the reason that the fill has just been made and it is not advisable to place the permanent pavement at this time. There is a ready supply of white cedar which is highly satisfactory for plank floors. S. A. Jubb is Assistant Harbor Engineer.

SAN FRANCISCO, CALIF.—In the work under the direction of the Board of State Harbor Commissioners, there is a variety of pavements in driveways, such as wood blocks treated with carbolineum, ohia (Hawaiian hardwood) blocks, plank (both Oregon fir and cedar), brick and bituminous concrete. No standard has yet been adopted, as experiments are being made with the idea of selecting the material best suited for the purpose. Until a few years ago the timber piers were planked, sometimes using a single 4- or 6-in. thickness of Oregon fir and in other cases using a 3-in. layer of Oregon fir covered by a 3-in. wearing surface of cedar, the idea being to utilize the superior durability of cedar. But the life of both of these materials was short, the Oregon fir lasting usually one year and the cedar about two years.

For the new timber piers a substitute for planking is being sought and some of the structures now being built are to have the driveways of creosoted wood blocks or untreated hardwood. This pavement is laid on planking, the space between the planks being wedged and the planks being covered with roofing felt upon which is laid a 1-in. sand cushion to carry the blocks. On other timber piers experiments are being made with bituminous concrete, and good results are expected. But experience with both these materials has been too short to allow of forming a definite opinion as to their merits. On concrete piers, wood blocks used on a sand cushion have proved satisfactory for the driveways. On some of the later concrete piers, brick was used as a pavement in the driveways, but did not prove satisfactory on account of the inferior quality of the material.

On the newer piers, the pavement in the cargo spaces on each side of the driveway is of asphalt laid directly on the concrete slab in the case of concrete piers and on a wooden plank foundation in timber piers. This has proved satisfactory, as there is not much heavy trucking on this portion of the pavement. It may be considered standard, as it has been found reasonable in first cost, easily repaired and durable. Jerome Newman, Assistant State Engineer, is Chief Engineer of the Board.

PORTLAND, ORE.—In the dock and warehouse construction by the city's Commission of Public Docks, all interior floors and driveways are of wood, except in a double-deck shed now nearing completion. Here the lower floor (100x526 ft.) has an 8-in. reinforced-concrete slab. The floors of all docks, except as noted above, consist of two layers of tongued-and-grooved flooring, the bottom layer in all cases being 3 in. thick. The top layer is 3 in. thick for the roadway or driveway and 1½ in. for the cargo space. For heavy teaming, the wooden floor soon becomes cut up, and it is probable that later the team driveway will be covered with a 2-in. layer of asphaltic cement. G. V. Hegardt is Engineer of the Board.

SEATTLE, WASH.—The experience with roadway surfacing in warehouses and freight sheds has been too short

to permit of conclusions of much value. For paving driveways in freight sheds and warehouses, there have been used ordinary asphalt pavement, plank roadways constructed of 4x12-in. Douglas fir, and plank roadways built of 4x4-in. Douglas fir laid as nearly as possible with the grain vertical.

On the newer work, where it is expected that the roadway will have to stand hard service, asphalt paving is being used. And where the plank roadway is sufficient, it has been found that the 4x4-in. planks laid with the grain vertical and the direction of the planks diagonal to the roadway is quite satisfactory. All of the roadways are laid on a foundation of 4x12-in. planks which are either carried on stringers or on sleepers embedded in the fill.

The floor surfaces of the older freight sheds consist merely of 4x12-in. Douglas fir plank. On the newer work, however, there is a 1¾-in. wearing surface of vertical-grain, air-seasoned Douglas fir lumber on top of the rough planking. This latter construction promises to be durable and presents a smooth surface for trucking. J. R. West is Acting Chief Engineer for the Seattle Port Commission.

New York, N. Y.—The general practice in New York has been to place a 4-in. wood sheathing on top of the 4-in. wood deck on all pier structures, whether built by the city or by private interests. The wood-wearing surface was very unsatisfactory on account of its unsanitary condition, its wear and grinding and its extremely high cost in renewals every few years. About nine years ago, therefore, the Department of Docks, under the direction of C. W. Staniford, Chief Engineer, tried the use of a concrete deck.

It was assumed that a concrete deck alone might take care of team traffic, and the new construction consisted of a reinforced-concrete slab with a smooth hard-finished concrete surface. This was in the nature of an experiment, to find out definitely whether the concrete would do the work and save from $1.25 to $2.50 per sq.yd. in total expense. After a few years' trial this proved unsuccessful, owing to the fact that with teams turning around on the concrete, the hard top surface became broken, after which further disintegration and grinding took place. It soon became evident that some tough surface was necessary to protect the reinforced-concrete deck slab.

The plan adopted was to place a top surface of asphalt on the concrete decks, using 1¾ in. of binder and ½ in. of top surface. The results have been satisfactory under very heavy traffic.

The asphalt surface on the lower decks of the Chelsea piers, where the traffic is severe, has shown excellent lasting qualities. The second story, or promenade deck, of these piers is covered with ½ in. of rock asphalt laid on the reinforced-concrete slab on the I-beams. This also has given excellent service, although the floors are used generally for freight-handling purposes as well as the reception of passengers.

Paving for Garages and Wagon Sheds

For garages, municipal wagon sheds, fire-engine stations, etc., where heavy equipment is handled, four out of six firms favor a paving of reinforced concrete. One favors creosoted wood-block paving on a concrete base, and one favors wood-block paving for the interior driveways and concrete for the remainder of the floor.

News of the Engineering World

Improvement of the Connecticut River at Windsor Locks

Since the change of control of the Connecticut River Co., last year, and a consequent reorganization, it has been generally known that some further development of the river for power and navigation would be undertaken at Windsor Locks, Conn. On account of the many inquiries which continue to be made as to the status of the work at this point and the reasons for lack of development, it is of interest to quote the following statement made to us by Charles H. Tenney, of Springfield, Mass., President of the Connecticut River Co.:

The Connecticut River Co. has occupied that section of the Connecticut River for 94 years, has built and today maintains a dam across the Connecticut River, which has been there for thirty-odd years and backs the water up to the City of Holyoke, a distance all told of approximately 20 mi. The dam is built at the head of the rapids and the water carried in a canal about 6 mi. to the foot of the rapids, where the power is developed by different factories, and the canal gives a measure of navigation.

For at least 20 years, Springfield and Holyoke navigationists have been demanding that the Windsor Locks Co., which was chartered among other things, for the purpose of improving navigation on the Connecticut River, change the location of its dam from the head to the foot of the rapids. People of these two cities pointed out to the Windsor Locks Co. that they could well afford to do this because they would develop for power the entire flow of the river, where today, owing to the narrowness of the long canal, they only develop about one-tenth. But for some reason not known to the present owners of the Connecticut River Co., the officers and directors of that company refused to do anything, although there was no question as to their full authority to build a new dam at the lower point.

About 1906 it was decided by act of Congress that no dams should be built in navigable streams without the consent of Congress and, when the control of the Connecticut River Co. was taken over by a syndicate of Connecticut and Massachusetts people interested in navigation, we found Congress quarreling over what the terms of that consent should be. So that today we are in a position of being ready and willing to go ahead with the development, which would add greatly to the prosperity of this section, we believe, but are unable to do anything owing to the fact that Congress cannot determine in its own mind what conditions, if any, to impose on us for moving our dam downstream. The situation looks to the writer a good deal as it would be if the City of Springfield should say to its people, "You shall build no more buildings in the City of Springfield until we have formulated rules and regulations for them" and then should refuse to pass any rules and regulations or even have the matter up for consideration.

Completing the New Mississippi River Bridge at St. Louis

The Municipal Bridge at St. Louis, built across the Mississippi River a few years ago by the city as a "free" highway and railway bridge to avoid the tolls on the other

bridges, is to be made available at last by the construction of the approach on the east, or Illinois side. At present there is no access to the bridge.

The new east-end railway approach will be 2.3 miles in length, with seven spur connections to the four lines of railway which it will cross (Mobile & Ohio, Illinois Central, St. Louis Terminal, and Alton & Southern). The highway approach will be 0.7 mile in length, with connections for street and interurban electric lines. There will be used about 24,000 tons of steel and 35,000 cu.yd. of concrete. The structure will have eight spans of 200 ft. and over, including two of 296 ft. and one of 276 ft. for crossing the railways. The remainder of the work will be girder spans of 45 to 75 ft.

For the piers of the large spans there will be 18 caisson foundations sunk through sand, quicksand and loose material to good boulder gravel at a depth of about 45 ft. The pedestals for the viaduct towers will have groups of concrete piles under the footings. The piles are being manufactured under contract, as it was not desired to establish a special plant for these.

With the exception of the concrete piles and the fabrication of the steel, all the work will be done by the city by day labor. This was agreed upon when the bond issue for the work was to be voted on in November last, the city promising that only union labor would be employed. Under the city's charter a contract could not be let where union labor was specified, and in order to keep their promise the city authorities decided to handle the work themselves, through the Board of Public Service. Clinton H. Fisk is Engineer of Construction for the Board, and a complete working force and equipment are being organized.

The consulting engineers are Boller, Hodge & Baird, of New York, and Brenneke & Fay, of St. Louis, act as their resident engineers. These firms occupied the same positions in regard to the bridge proper and its western (St. Louis) approach. Work is in progress on the substructure and it is expected to have the eastern approach completed and the bridge opened for traffic by October, 1916.

Another Wood-Block Bridge-Floor Fire

In the early morning of Apr. 29 the creosoted wood-block pavement and timber-flooring system of the Connaught Bridge, Vancouver, B. C., caught fire from some cause at present unknown, but attributed either to sparks carried on a high wind, or incendiarism.

FIRE ON FLOOR OF CONNAUGHT BRIDGE, VANCOUVER

The bridge is a steel structure crossing False Creek, and with approaches and drawspan it has a length of 4091 ft. It was built to the designs of Waddell & Harrington, completed three and one-half years ago and cost altogether $740,000.

The fire occurred at a point about 3000 ft. to the north of the drawspan, and the fierce heat resulting from creosoted timber and a 60-mi. gale caused the complete col-

lapse in about two hours of a 90-ft. girder span, as shown in the view, and serious buckling and sagging of two adjacent similar spans. About 150 ft. of the bridge between the collapsed portion and the draw was thrown out of alignment, and the latter became so jammed as to prevent its opening.

The floor system consists of transverse wood joists bearing on the steel floor-beams, 4-in. planking and tar-paper forming the bed for the wood-blocks; all timber is creosoted.

No fire hydrants were placed on the bridge and a fire-boat was not available, and the location of the outbreak made the fire difficult to fight. It was finally confined by cutting gaps in the flooring. The damage is estimated at about $75,000, the cost falling on the city, as no insurance was carried.

Curiously, at about the same time fire also broke out in the flooring of Granville Street Bridge, a similar structure lower down the creek. This was controlled before it assumed serious proportions.—H. F. JEFFERSON, Vancouver, B. C.

Los Angeles Annexes More Territory

Los Angeles at its primary election of May 4 voted (37,000 for, 24,000 against) to annex what is called the San Fernando District, and in so doing arrived at a decisive conclusion as to the handling of the vexatious problem of disposal of the surplus waters of the Los Angeles Aqueduct. The San Fernando District comprises nearly all the tillable area of the San Fernando Valley (exclusive of three small incorporated cities), with an area of 103,800 acres, and lies to the northwest of Los Angeles and below the aqueduct outlet. In the same referendum, a smaller section to the southwest, called the Palms and having an area of 8000 acres, was likewise annexed (38,000 for, 20,000 against) as a part of the plan for water distribution. The area of the city before annexation was 68,877 acres, or 107.6 sq.mi.

The newly incorporated territory by the terms of annexation agrees to assume its proportionate share of the aqueduct, harbor and power-development bonds. In addition, the larger area has already formed itself into an assessment district and voted $2,606,000 worth of bonds for the building of an irrigation system to receive its supply from the aqueduct. This system, upon completion, it has been agreed to deed to the city.

The initial rate set for the sale of irrigation water is one cent per miner's inch per hour, which amounts to about $11.68 per acre per annum continuous flow to 7½ acres. The rate, however, is not a fixed one but must be set annually by the City Council. The present rate may therefore be either raised or lowered to raise the exigencies of the city's finances. These terms, at first reading, may appear onerous on the new territory. However, in the great influx of population to Southern California during the past ten years the San Fernando Valley until the present time has had little development. Waterless and in a semi-arid region, its rich and virgin soil has brought forth only scanty crops from dry farming. Water has been the one thing needed. Now, with the assurance of an abundant supply, the region has the certainty of rapid development.

The system to be built comprises 317 miles of steel mains ranging from 8 in. to 52 in. and aggregating a weight of 25,000 tons, which has been designed and probably will be built under the direction of William Mulholland, designer and constructor of the Los Angeles Aqueduct and Chief Engineer of the Los Angeles Department of Public Service. The estimate of time for completion is two years, and it is expected that work can be undertaken almost immediately. To provide water for a part of the district during the coming season (the vote of May 4 was anticipated) a temporary ditch 7 miles long has been built along one side of the valley. This earthen unlined ditch, with a capacity of 8000 miner's inches, was begun Apr. 1 and will be completed within the next ten days.

The election has established the principle that only such urban territory, contiguous to the city, as will build its own distribution system and annex itself to the city shall enjoy the use of the water. It has required more than five years to bring about a condition of the public mind which would make possible a comprehensive plan of distribution and provide a source of funds which would make the consummation of the plan a reality.

Furthermore, it is the first step toward the ultimate consolidation of the city and county of Los Angeles. After supplying the newly annexed districts there will still remain a surplus of approximately 8000 miner's inches for territory still outstanding. With the terms of use now fully and finally fixed, it is expected that further annexations will follow until all the water is applied to the land when the boundaries of such territory will comprise the limits of the new city coincident with those of the county. In this way, then, Los Angeles sees the method of building, operating, policing and maintaining the sanitary conditions surrounding her great water-works system.—BURT A. HEINLY, Los Angeles, Calif.

A Bill Creating a Purchasing Department for New York City, passed by the 1915 Legislature, has been vetoed by Mayor Mitchel. It is said that the mayor approved the general object of the bill but believed it to have been so imperfectly drawn that it would have defeated its purpose.

Gasoline Fire Engines for the New York City Fire Department will probably ultimately replace all the steam pumping-engine units now operated outside the high-pressure zones. Six gasoline units for new companies which are to be organized were ordered by the Fire Department on Apr. 13, for a total price of $43,338, deliveries to be made within 100 days.

Chlorine Gas for Water Disinfection at Terre Haute, Ind., is being produced from rock salt by means of electrolytic cells and is discharged directly into the pump well by means of a plant put into operation on May 8. The plant was installed by, and at the expense of, the Chlorida Process Co., Chicago.; the cells, process and auxiliaries are protected by patents granted and pending.

Irrigation Work in Peru on 90,000 acres of desert land near Callao has been arranged for by a contract between Breitung & Co., Ltd., of New York, and the Government of Peru. Under the contract the Peruvian Government will issue bonds to the amount of $10,000,000, maturing in 30 years, to pay for the work. The lands to be irrigated under the contract are to be sold to purchasers on 20-year payments.

Low Water in Lake Superior has reduced by 6 in. the safe draft of vessels passing through the Sault canals. The Lake Carriers' Association on May 8 advised vessel owners not to load deeper than 19 ft. 3 in. in taking cargoes at the head of the lakes for Lake Michigan. Ore boats going to Lake Erie ports are loading only to 18 ft. 9 in. The reduction in drafts reduces the cargoes of the large ore and coal boats by 200 to 500 tons.

Channel Changes at St. Paul, Minn., have been recommended in the report of the board of army engineers ap-

pointed to investigate the application of the city to make certain changes in the channel of the Mississippi incident to the construction of the new Union Station, described in "Engineering News," Mar. 11, 1915, p. 488. The report will now go to the Secretary of War, and upon his approval, authority will be granted the city to proceed with the work.

The Birmingham (Ala.) Building Department is to be made a part of the City Engineer's office, instead of being an independent bureau, as heretofore. This action was decided upon by the Board of City Commissioners following an investigation of the old-wall collapse which killed several people on Apr. 21, 1915 ("Engineering News," Apr. 29, p. 846). The City Engineer, Julian Kendrick, will now be in charge of the building and plumbing departments as bureaus of his own office.

Cast-Iron Pipe 88 Years Old—The Philadelphia Water Bureau has recently removed about 275 ft. of 6-in. cast-iron pipe that was laid in 1827 on Locust St. from 7th to 8th. The pipe was in service up to the time of its removal and was taken away in order to make room for the construction of a large sewer. It is supposed that the pipe was made in England. It was in 9-ft. lengths and was evidently cast on its side, as the thickness varies from ½ to ⅝ in. The iron shows no evidence of deterioration, and except for the necessity of making way for the sewer there is no reason why it should not continue in service indefinitely. The interior of the pipe was heavily coated with tubercles, some of these projecting 1½ in. The effective carrying capacity of the pipe, however, had not been reduced to an extent making it seriously inefficient for ordinary distribution service.

The Boston Board of Health, following the example of the Massachusetts Board of Health, will be superseded by a single Health Commissioner, provided for by a recently passed city ordinance. Mayor Curley has secured Dr. Richard H. Creel, of the Federal Public Health Service, to fill the newly created position. Dr. Creel will reorganize the department into divisions of medicine, child hygiene, sanitation, food inspection, laboratory work and vital statistics, records and accounts. The ordinance requires that each division shall be in charge of a deputy commissioner, who shall be appointed by the Health Commissioner, each to be a person of recognized standing in his profession or occupation and an expert in the duties that may devolve upon him. Dr. Creel will receive a salary of $7500 as commissioner. He will go to Boston on an indefinite leave of absence from the Public Health Service.

The Operation of the National Transcontinental Ry. of Canada between Moncton, N. B., and Winnipeg, Man., was taken over by the Canadian Government on May 1. This line was constructed by the government with the understanding that it would be leased to and operated by the Grand Trunk Pacific Ry. in connection with its line running west from Winnipeg to the Pacific Coast. The Grand Trunk Pacific, however, raised numerous objections to fulfilling the contract and finally definitely refused to lease the road. The Canadian Government also took over the Lake Superior Division of the Grand Trunk Pacific between Superior Junction on the main line and Fort William. A service of two trains a week each way between Moncton and Winnipeg will be given at first, to be increased as soon as the amount of traffic requires it. Sections of the road between Moncton and Lewis, Que., and between Winnipeg and Fort William are already in operation.

The Earnings of the Panama Canal during the eight months since it was opened to traffic have not been enough to pay its expenses of operation and maintenance, according to the "Canal Record." In round figures the total expenses charged to operation and maintenance during the eight months was $2,600,000. The receipts from tolls in that time were $2,334,000. At the present time, however, the canal is earning a small surplus over its operating expenses. The tolls earned during March amounted to $561,000, and a close estimate of the cost of operation and maintenance is $425,000. It is noted also that included in the cost of operation and maintenance since July 1 is $1,012,000 for dredging in the Culebra Cut and over $100,000 for dredging in the Atlantic entrance. The next largest item in the cost of operation and maintenance is the proportion of the cost of the civil government, health department and administration charged to the canal, amounting since July 1 to nearly $900,000. This item exceeds the entire cost of maintaining and operating the canal itself with its structures and appurtenances.

A Privately Owned Manufacturing District, equipped with modern facilities and having numerous tenant companies, is one of the features of the modern industrial development of Chicago. The Central Manufacturing District is the name of the organization, which is managed by trustees representing the interests which own and control the stock yards, the Chicago Junction Ry., large warehouse and storage facilities, and the new produce market and railway terminal noted in "Engineering News," Apr. 29. The district has an area of about 300 acres, bounded by Morgan, 35th and 39th St. and Ashland Ave. There are at present about 200 tenants, occupying sites of varying area and having buildings in size and design adapted to their special requirements. Paved streets are laid out, and sewer and water mains are provided. Railway spurs connecting with the Chicago Junction Ry. serve all the several industries, thus eliminating team service to freight terminals, while those along the river (including lumber yards and warehouses) have wharves for lighterage or steamer service. At 26th St. the same interests have a waterfront freight house 100x800 ft. for steamer and barge service. J. A. Spoor and Arthur G. Leonard are the trustees of the Central Manufacturing District.

A Filter Plant to have a capacity of 400,000 gal. per hr. is being constructed by the Town of Brookline, Mass., a suburb of Boston, the work being done by day labor under the direction of the Superintendent of Water-Works, F. F. Forbes. Brookline, with a population in excess of 30,000, takes its supply from wells and filter galleries along the Charles River and, as is the case with most towns having ground-water supplies, has begun to experience trouble from the increasing amounts of iron and manganese. Complaints were numerous, and it seemed probable that the supply might ultimately have to be abandoned. As the result of experiments carried out by the Water Board, under the direction of R. S. Weston, Consulting Engineer, and in accordance with his recommendations, a deferrization plant is being constructed, to consist of an aërator, coke tricklers, coagulating basin, covered slow sand filters and a covered reservoir. The tricklers are cylindrical tanks, 31 ft. in diameter, six in number, which will probably be built of reinforced concrete and filled with coarse coke. The coagulating basin and the six filters form two groups of four and three chambers, respectively, each being about 69x110 ft. and 12.5 ft. deep, and will be of concrete with groined floors and roofs. The filters will contain about 4 ft. of sand. The covered reservoir will be about 110x275 ft. in plan, 19.5 ft. deep, of concrete with groined floor and roof, and it will have a capacity of about 4,000,000 gal., which is about one day's supply in times of maximum draft. The experiments showed that after the aëration a large part of the iron was retained by the coke tricklers, from which it can be removed in large measure by flushing. They serve, therefore, as roughing filters which will greatly reduce the frequency with which the sand filters will have to be cleaned.

PERSONALS

Mr. Chas. H. Folwell, Secretary of the New Jersey State Water Supply Commission, has resigned, effective June 1, after seven years' service.

Mr. Robert S. Brown, graduate of Trinity College, Durham, N. C., has opened an office in the Legal Bldg., Asheville, N. C., for the general practice of civil engineering.

Mr. H. W. Miller, recently Assistant to the President of the Southern Ry., has been appointed Vice-President. He has been in the service of the Southern Ry. for about 30 years.

Mr. John W. Moran, recently Chief Engineer of the Watuppa pumping station, Fall River, Mass., has been appointed Superintendent of the Water-Works by the Fall River City Water Board.

Mr. Frederick A. Caldwell has resigned as Assistant City Engineer of Providence, R. I., after having served 19 years in the Water Department and the City Engineer's office of that city.

Mr. E. H. Packard has been appointed City Engineer of Bloomington, Ill., under the new commission form of government. He succeeds Mr. Elmer Folsom, who has held the position for several years.

Mr. E. V. Frothingham, Commissioner of Public Works of the Borough of Manhattan, New York City, has resigned, effective June 1. He was appointed Feb. 15, 1910, by Borough President George McAneny.

Messrs. O. L. Remington, General Manager, and H. P. McColl, Chief Engineer, of William McLean & Co., Engineers and Manufacturers, of Melbourne, Australia, are touring the industrial centers of this country and England.

Mr. W. H. Winterrowd, M. Am. Soc. M. E., Mechanical Engineer of the Canadian Pacific Ry. at Montreal, Que., has been appointed Assistant to the Chief Mechanical Engineer of the Canadian Pacific, with headquarters at Montreal.

Mr. J. K. McNellie, recently Superintendent of District No. 3, of the Canadian Pacific Ry., has been appointed General Superintendent of the Intercolonial Ry., with headquarters

at Moncton, N. B., succeeding Mr. F. A. Brady, resigned, as noted elsewhere.

Sir Thomas George Shaughnessy, President of the Canadian Pacific Ry., has been elected an Honorary Member of the Institution of Civil Engineers of Great Britain. Although noted as a railway executive, he has never been connected with the engineering department of a railway system.

Mr. F. A. Brady, recently General Superintendent of the Intercolonial Ry. of Canada, has been appointed General Superintendent of the National Transcontinental Ry. between Quebec and Winnipeg, and the Lake Superior branch between Fort William and Superior Junction, the headquarters of Cochrane, Ont.

Mr. Thomas A. Edison, M. Am. Inst. E. E., has been presented with the gold medal of the American Civic Forum "not for any particular achievement, but for distinguished services and great scientific achievements, and in recognition of a great career which has a place among the very highest in the roll of human history."

Mr. Edward W. Parker, M. Am. Inst. M. E., has resigned as Coal Statistician of the Division of Mineral Resources, U. S. Geological Surevey, to accept a position with the anthracite mining companies. He has been Coal Statistician of the Geological Survey since 1890 and Chief of the Division of Mineral Resources since 1908.

Mr. William Gerig, M. Am. Soc. C. E., Vice-President and Chief Engineer of the Pacific & Eastern Ry. and Consulting Hydraulic Engineer of the Spokane, Portland & Seattle Ry., at Medford, Ore., has resigned to take a position with the Northern Pacific Ry. in connection with the work on the Union Station at St. Paul, Minn.

Mr. J. M. Preston has resigned as City Engineer of Dallas, Tex., to open an office in Dallas as consulting engineer. He has been City Engineer for the past seven years, and under his supervision several large engineering undertakings have been carried out, including the White Rock reservoir and a large amount of street-improvement work.

Mr. S. D. Clinton, Assoc. M. Am. Soc. C. E., recently Division Engineer of the Idaho State Highway Commission at Pocatello, Idaho, has been appointed Assistant Field Engineer of the Interstate Commerce Commission, Division of Valuation, Fifth District, with headquarters at San Francisco. Mr. Clinton will have charge of a party in northern Montana and succeeds Mr. C. N. Kast, M. Am. Soc. C. E., who has been promoted.

Mr. H. D. McCaskey, M. Am. Inst. M. E., Mining Geologist of the U. S. Geological Survey, has been appointed Chief of the Division of Mineral Resources to succeed Mr. Edward W. Parker, M. Am. Inst. M. E., who has resigned, as noted elsewhere. Mr. McCaskey has been Geologist of the Geological Survey, and from 1900 to 1903 was Mining Engineer of the Philippine Bureau of Mines, and was Chief of that bureau from 1903 to 1906.

Mr. Adelbert A. Weiland, M. Am. Soc. C. E., has been appointed State Engineer of Colorado. Mr. Weiland has been in private practice for the past five years, previous to which he was Hydraulic Engineer for the Colorado Fuel & Iron Co. Before that he was with the Water Resources Branch of the U. S. Geological Survey. In his new position he will have entire charge of the irrigation problems of Colorado and will also retain his private practice as a consulting engineer, with rooms in the Thatcher Bldg., Pueblo, Colo.

Mr. Fred. M. Rutter, Assoc. M. Can. Soc. C. E., recently Resident Engineer of the Canadian Pacific Ry. at West Toronto, Ont., has been promoted to be Assistant Superintendent of the Ontario Division of the Canadian Pacific, with headquarters at Toronto. Mr. Rutter entered the engineering department of the railway in 1902 and was engaged for a time in the construction of the Toronto-Sudbury line. Later, he was Resident Engineer at Woodstock, N. B., and since 1910 has been stationed at West Toronto and Montreal.

Major-General George Washington Goethals, M. Am. Soc. C. E., has been elected an Honorary Member of the Institution of Civil Engineers of Great Britain. In commenting on his appointment, London "Engineering" says: "Colonel Goethals has proved himself a captain of industry in the most literal sense of the term. It has been his task to direct the canal forces and to stimulate them to fine performances. Though he cannot, like the leader in commercial affairs, point to glowing financial reports, everybody is conscious that good work has been accomplished at Panama."

Mr. Ross A. Frederick has been appointed City Engineer of Sault Ste. Marie, Mich. Mr. Frederick has been engaged as designing and constructing engineer in the railroad, industrial and municipal field for 14 years. For three years he was Assistant Construction Engineer with the Lake Superior Corporation, for two years Assistant Bridge Engineer of the Southern Ry., and for three years Office and Field Engineer with the Grand Trunk Ry. For four years he was with the General Motors Co. as Office Engineer and Superintendent of Construction, and for the past two years has been Office Engineer of the Algoma Central Ry.

Mr. J. E. Stephens, Chief Engineer of the Missouri Pacific-Iron Mountain Ry. system, has been promoted to be Assistant to the President, succeeding Mr. Alexander Robertson, who has been made Vice-President in Charge of Operation. Mr. Stephens graduated from Stevens Institute of Technology at Hoboken in 1878, and entered the railway service in 1880 with the Oregon-Washington R.R. & Navigation Co. Since then he has been in railway-engineering work in various parts of the United States and Canada, South Africa and Mexico. From 1911 to 1912 he was Assistant Chief Engineer of the Missouri Pacific and the St. Louis, Iron Mountain & Southern Rys., and since 1911 he has been Chief Engineer.

Mr. E. A. Hadley, recently engaged on special engineering investigations for the President's office of the Missouri Pacific Ry. and the St. Louis, Iron Mountain & Southern Ry., St. Louis, Mo., has been appointed Chief Engineer of the railway to succeed Mr. J. E. Stephens, promoted, as noted elsewhere. Mr. Hadley was born in Lowell, Mass., in 1879, was educated at the public schools of Lowell and was graduated from the high school there in 1897. He was an apprentice to Smith & Brooks, Civil Engineers, Lowell, Mass., from 1897 to 1901, and in 1901 joined the engineering staff of the Boston & Maine R.R. as draftsman. Later, he was promoted to be Assistant Engineer and Resident Engineer. In June, 1910, he went to the Missouri Pacific Ry. as Engineer of Design.

OBITUARY

Calvin Whiteley, Jr., former Chief Engineer of the United Railways & Electric Co., Baltimore, Md., died May 3 from apoplexy.

Albert Lloyd Hopkins, Assoc. M. Am. Soc. C. E., President of the Newport News Shipbuilding & Dry Dock Co., of New York City, was among those lost when the "Lusitania" sank on May 7.

Gustav Wilke, one of the earliest contractors for "skyscraper" construction in Chicago and a building contractor of national reputation, died Apr. 24 at his home in Chicago. One of his first large contracts was the building of the Texas state capitol at Austin. He was a member of the contracting firm of Wilke & Hoag, Chicago. He is survived by a widow and two children.

Thomas A. Leach, for many years Superintendent of the Blackstone Valley Division of the Worcester Consolidated Street Ry., and one of the pioneer street-railway men of New England, died at his home in Worcester, May 5. He was born in Boston in 1852, and was educated in the public schools of that city. He took up street-railway work at the very beginning of the industry.

Edmund K. Turner, M. Am. Soc. C. E., a well-known consulting engineer, of Boston, died at the Corey Hill Hospital in Brookline on May 6. He was born at Marblehead, Mass., in 1848, and was graduated from the Massachusetts Institute of Technology in 1870. After several years with the Fitchburg R.R. he entered private practice and was connected with many important enterprises. Mr. Turner was a member of the Boston Society of Civil Engineers.

Alexis H. French, M. Am. Soc. C. E., for many years Town Engineer of Brookline, Mass., died May 3 at his home in Brookline. He was a graduate of the Massachusetts Institute of Technology, class of 1873, and was 64 years of age. He had been the head of the Brookline engineering department since its establishment in 1894, and previously had done engineering work for the town for many years. He was a member of the Boston Society of Civil Engineers, the Boston Engineers Club, the Massachusetts Highway Association and many other associations in Boston and vicinity. He is survived by a widow, a brother and a sister.

Lindon Bates, Jr. (son of Lindon Wallace Bates), an American engineer connected with the Red Cross work in Belgium, was among those lost in the sinking of the "Lusitania" on May 7. He was born in Portland, Ore., in 1883, was educated in England and at Yale University, from which he was graduated in 1902. He had been engaged in engineering work at Galveston, on the New York State barge canal and in Denver, Colo. Later he traveled extensively in Siberia and South America. He was Vice-President of the Bates Engineering Co. of New York City. He served on engineering boards for

New York City in connection with the Catskill Aqueduct and other works, and was a member of enginering societies in Belgium and other countries.

William Cornell Jewett, M. Am. Soc. C. E., died at Santa Paula, Calif., May 2, 1915. During his early life he was connected with the construction of the Cincinnati Southern R.R., the Alabama Great Southern R.R. and the New Orleans & Northeastern R.R. In 1891 he resigned as Engineer of Maintenance-of-Way of the Cincinnati Southern and Alabama Great Southern railways to become Chief Engineer of construction of a 30-mile extension of the Cleveland, Lorain & Wheeling R.R. at Cleveland. On the completion of this work in 1896 he returned to Cincinnati and was associated with Gustave Bouscaren as Resident Engineer on the construction of the new Cincinnati water-works. He was retained in this capacity until the completion of the work in 1909. In 1910 he was appointed Chief Engineer in charge of the construction of the new General Hospital in Cincinnati. His work on this $4,000,000 group of buildings had just been completed and he was on a pleasure trip to California. Mr. Jewett was 61 years old. He is survived by a widow, a daughter and a son.

William Pond Harris, of Johnson City, Tenn., died at his home Apr. 24, 1915, from pneumonia. He was born in Worcester, Mass., Feb. 12, 1846, and was graduated from Rensselaer Polytechnic Institute, class of 1866. He began work as an engineer, being employed at various times upon the construction of the Hoosac Tunnel, improvement of Prospect Park, Brooklyn, and with the Northern Pacific Ry. He laid out Riverside suburb, Chicago. For a brief period he engaged in contracting, but later entered the railway service as an operating officer, being employed successively with the Delaware, Lackawanna & Western R.R., the Chesapeake & Ohio Ry. as Superintendent, the Louisville & Nashville as Division Superintendent, the Baltimore & Ohio Ry., the Atchison, Topeka & Santa Fé Ry., the Charleston, Cincinnati & Chicago R.R., and with the Plant System, now the Atlantic Coast Line Ry. In 1898 he moved to Johnson City, Tenn., and established the Harris Manufacturing Co., a successful wood-working industry. Mr. Harris was a man of sterling character, great energy and of high intellectual attainments, and is mourned by a large circle of friends. [Contributed by Orlando K. Morgan, Johnson City, Tenn.]

Dr. F. S. Pearson, M. Am. Soc. C. E., Consulting Engineer, of New York City, and the promoter of many engineering enterprises in South America and other foreign countries, was among those drowned when the steamship "Lusitania" was sunk off the Irish Coast on May 7. Mrs. Pearson, who accompanied him, is also reported to have been lost. Fred Stark Pearson was born in Lowell, Mass., July 3, 1861. He was graduated from Tufts College in 1883. From 1879 to 1880 he was Instructor in chemistry at the Massachusetts Institute of Technology, and from 1883 to 1886 was Instructor in mathematics and applied mechanics at Tufts College. He was a mining engineer in the United States and South America from 1886 to 1888, returning to this country to become Manager of the Somerville (Mass.) Electric Light Co. Later he was Chief Engineer of the West End Street Ry. Co. of Boston, of the Dominion Coal Co., of the Metropolitan Street Ry. Co., New York, from 1894 to 1899, and since then has been a consulting engineer and promoter of world-wide reputation. He was President and Director of the Barcelona (Spain) Traction, Light & Power Co., Ltd., the Mexican Tramways Co., the Rio Janeiro Tramway, Light & Power Co., the Mexican Light & Power Co., Ltd., the Mexican Northwestern Ry. Co. and the Sao Paulo Tramway, Light & Power Co. Mr. Pearson was a member of the Institution of Civil Engineers, of the American Institute of Electrical Engineers, the American Institute of Mining Engineers, the American Society of Mechanical Engineers, American Society of Civil Engineers and many other scientific associations and clubs.

ENGINEERING SOCIETIES

COMING MEETINGS

INTERNATIONAL RAILWAY FUEL ASSOCIATION.
May 17-20. Annual meeting at Chicago. Secy., C. G. Hall, 922 McCormick Bldg., Chicago.

SOUTHWESTERN ELECTRICAL AND GAS ASSOCIATION.
May 19. Annual convention at Galveston, Tex. Secy., H. S. Cooper, Dallas, Tex.

NATIONAL MACHINE TOOL BUILDERS' ASSOCIATION.
May 20-21. Spring meeting at Atlantic City, at Hotel Marlborough-Blenheim. Secy., Charles E. Hildreth, Worcester, Mass.

MASTER BOILER MAKERS' ASSOCIATION.
May 25-28. Annual meeting in Chicago. Secy., H. D. Vought, 95 Liberty St., New York City.

AMERICAN SUPPLY AND MACHINERY MANUFACTURERS' ASSOCIATION.
June 3-4. Annual convention at Bellevue-Stratford, Philadelphia, Penn. Secy., F. D. Mitchell, 1510 Woolworth Bldg., New York City.

NATIONAL DISTRICT HEATING ASSOCIATION.
June 1-3. Annual convention at Hotel Sherman, Chicago. Secy., B. L. Gaskill.

NATIONAL CONFERENCE ON CITY PLANNING.
June 7-9. National Conference in Detroit, Mich. Secy., Flavel Shurtleff, 19 Congress St., Boston, Mass.

AMERICAN RAILWAY MASTER MECHANICS' ASSOCIATION.
June 9-11. Annual convention in Atlantic City. Secy., Jos. M. Taylor, Karpen Bldg., Chicago.

MASTER CAR BUILDERS' ASSOCIATION.
June 14-16. Annual convention in Atlantic City. Secy., Jos. M. Taylor, Karpen Bldg., Chicago.

OHIO SOCIETY OF MECHANICAL, ELECTRICAL AND STEAM ENGINEERS.
June 17-18. Semi-annual meeting at Toledo. Secy., Frank E. Sanborn, Ohio State University, Columbus.

SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION.
June 22-25. Annual meeting at Iowa State College, Ames, Iowa. Secy., F. L. Bishop, University of Pittsburgh, Pittsburgh, Penn.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.
June 22-25. Summer meeting at Buffalo, N. Y. Secy., Calvin W. Rice, 29 W. 39th St., New York City.

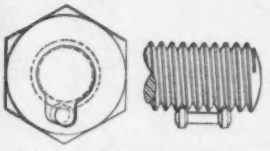
AMERICAN SOCIETY FOR TESTING MATERIALS.
June 22-26. Annual meeting in Atlantic City. Secy., Edgar Marburg, University of Pennsylvania, Philadelphia, Penn.

American Society for Testing Materials—The provisional program for the 18th annual meeting of the American Society for Testing Materials, to be held June 22-26 in Atlantic City, N. J., at the Hotel Traymore, has just been issued. In addition to the usual business meetings and committee reports, the following papers will be presented: "The Fusibility of Coal Ash," A. C. Fieldner, A. E. Hall and A. L. Feild; "Fatigue of Copper Alloys," Ernst Johnson; "Molten Zinc as a Reagent for Etching in the Microscopic Examination of Steel," Jesse L. Jones; "The Determination of Spelter Coating on Sheets and Wire," J. A. Aupperle; "Battery Zincs—Some Causes of Defective Service," Robert Job and F. F. White; "Elastic Limit," T. D. Lynch; "The Elastic Limit of Steel Determined by Transverse Test, and Its Relation to the Tensile Elastic Limit," W. K. Shepard; "Micrographic Determination of Surface Decarburization in Heat-Treated Steels," J. G. Ayers, Jr.; "The Relations among Maximum Strength, Brinell Hardness and Scierscope Hardness in Treated and Untreated Alloy and Plain Steels," R. R. Abbott; "Some Neglected Phenomena in the Heat Treatment of Steel," M. E. Leeds; "The Charpy Impact Test on Heat-Treated Steel," J. J. Thomas; "Some Reasons Why the Use of Heat-Treated Material Has Been Subject to Criticism," H. V. Wille; "Comparison of Physical Properties Obtained from Carbon Forging Steel When Water- and Oil-Quenched," C. D. Young; "The New Physical and Chemical Laboratory of the Pennsylvania Railroad Co. at Altoona," C. D. Young; "The Failure of Materials under Repeated Stress," H. F. Moore and F. B. Seely; "A Laboratory Method of Determining Pressure on Walls and Bins," J. Hammond Smith; "A Universal Strainometer of Simple Design," S. H. Graf; "Microstructure of Concretes, "N. C. Johnson; "The Effect of Finer Grinding and a Higher SO_3 Content upon the Physical Properties of Portland Cement," P. H. Bates; "An Investigation of the Strength and Elastic Properties of Concrete-Filled Pipe Columns," A. F. Holmes and F. W. Swain; "Some Abrasive Tests on Portland-Cement Mortars, to Select Aggregates and Mixes for Concrete Highways," H. S. Mattimore; "Method of Weight-Volumetric Proportioning of Concrete Aggregates in Testing," J. A. Kitts; "The Legal Interpretation of the Word 'Vitrified' as Applied to Ceramic Products," Edward Orton, Jr.; "Tests of Reinforced Gypsum Beams," W. A. Slater; "A Cylinder Friction and Lubrication Testing Apparatus," A. E. Flowers; "Some Experiments on Technical Bitumens," S. R. Church and J. M. Weiss; "A Fungus Bed Test of Wood Preservatives," Cloyd M. Chapman. As has been announced, the Seventh Congress of the International Association for Testing Materials, which was to have been held at Petrograd, Aug. 12-17, 1915, has been postponed. Members of the International Association will not be held liable for dues from and after the beginning of the current fiscal year, Jan. 1, 1915, "until the Council, after change in the political situation, takes further action." It is earnestly recommended that the American members of the International Association take no steps toward withdrawing from the International Association during this critical period in its history. The Secretary of the Amercan Society for Testing Materials is Edgar Marburg, Univ. of Pennsylvania, Philadelphia, Penn.

Appliances and Materials

Novel Nut Lock

A novel and effective lock for bolt nuts that are liable to work backward, shown in the accompanying sketch, is being made by the American Lock Nut Co., Pullman Station, Chicago. A channel with sloping floor is cut across the threads of an ordinary nut, and in this is slipped a small locking spool, or pin. As the nut is screwed on the bolt the pin rolls in contact with the bolt and loose in the deeper part of the groove; but with any backward motion the pin rolls up the floor and wedges against the threads. The pin cannot injure the latter, as it bears only on their flat surfaces.

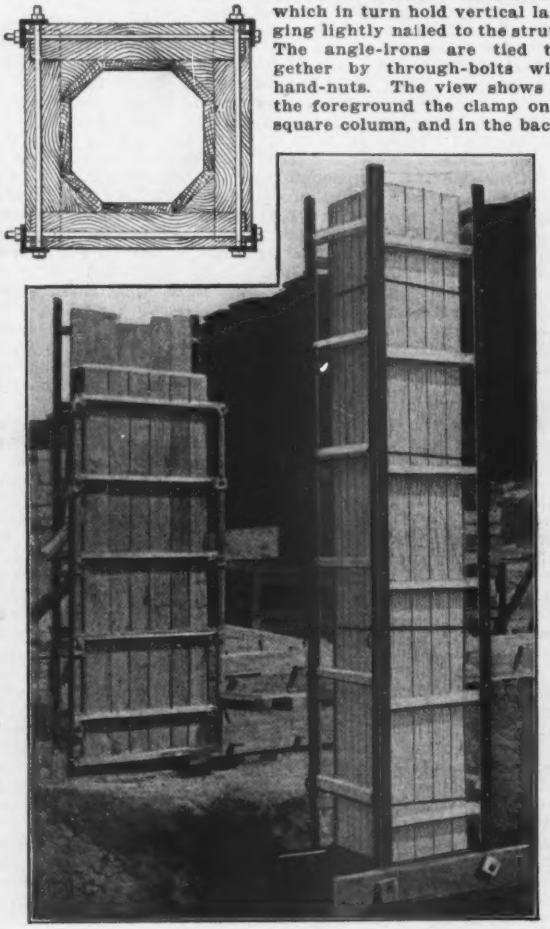


"Absolute" Lock Nut

* * *

Concrete Column Form Clamp

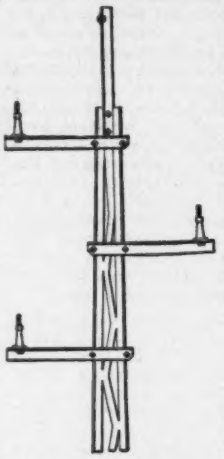
A new clamp for wooden forms for concrete columns has recently been put on the market by the New England Column Clamp Co., 6 Beacon St., Boston, Mass. The device, which is patented, consists of four ordinary angle-irons, which are used as the vertical corners of the form, acting as backing to hold wooden cross-struts, which in turn hold vertical lagging lightly nailed to the struts. The angle-irons are tied together by through-bolts with hand-nuts. The view shows in the foreground the clamp on a square column, and in the background on a rectangular wall column. The section in the upper corner shows the clamp as used on an octagonal column. Claims are made for superiority on account of saving in cost, both for manufacture and for placing, and for adaptability to all kinds of work. Because of the simplicity and rigidity of its construction, it can be used over and over again without very great deterioration.



Adjustable Column Form

Coast Survey Precise Level

To meet a growing demand for the very best type of precise leveling instrument, C. L. Berger & Sons, Boston, Mass., now manufacture and carry in stock the standard U. S. Coast & Geodetic Survey precise level (described in detail in "Engineering News," July 2, 1903). This is a dumpy level and has been used with much success by the Survey for the past 15 years. It was with one of these instruments that the precise-leveling records noted in "Engineering News" of Oct. 29 and Dec. 3, 1914, were made. The stock instrument of C. L. Berger & Sons is claimed to be built absolutely to the Coast Survey specifications, using nickel-iron and nickel-steel as called for, thus insuring the maintenance of adjustment under considerable changes in temperature. This precise level is guaranteed to pass the U. S. Coast & Geodetic Survey's inspection. The list price is $375.



Expanded-Steel Pole

* * *

Expanded-Steel Pole

The unique type of steel pole shown in the accompanying sketch has recently been developed by the Bates Expanded Steel Truss Co., South Lasalle St., Chicago. The body is formed by slitting the web of a special I-beam and pulling the flanges apart while hot. The top-arm and cross-arms are built of angles, which are bolted on. It is claimed that the prices approach the cost of wood poles for a given service.

* * *

Compact Traveling Mixer

The concreting outfit of the Municipal Engineering & Contracting Co., of Chicago, shown in the accompanying view, has compactness as one of its special features. Caterpillar supports are placed so close to the front axle that the machine can be turned in about its length. It has a cube concrete mixer of ¾-yd. capacity, mounted high enough to give a good slope to the 10-ft. distributing chute, which has two hinged sections (as shown) that can be swung down for depositing concrete at different distances. The charging hopper is operated by cable and drum and is raised to such a steep angle as to insure clear delivery of its contents into the mixer. The mouth is hinged on a crossbar of the double triangular frame, and this in turn is pivoted to the mixer frame. This rig, in lowering, shifts the hopper outward so that it drops to the ground and into position for convenient charging.

Compact Traveling Mixer

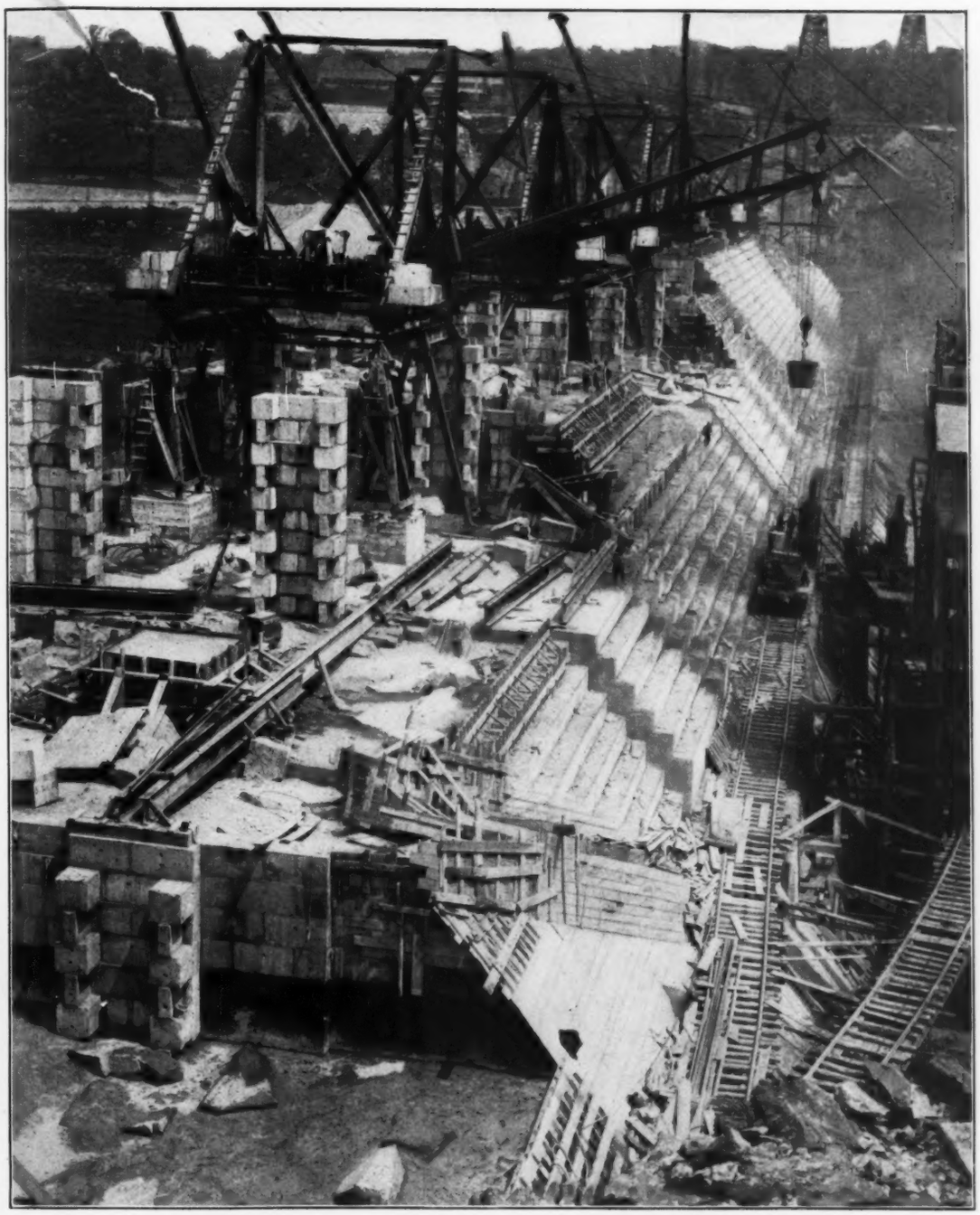


Fig. 7.—Downstream Face from West Hill on July 23, 1914.

Showing details of traveler-truck supports, contraction joints, downstream forms and service tracks.

Fig. 9.—Vie

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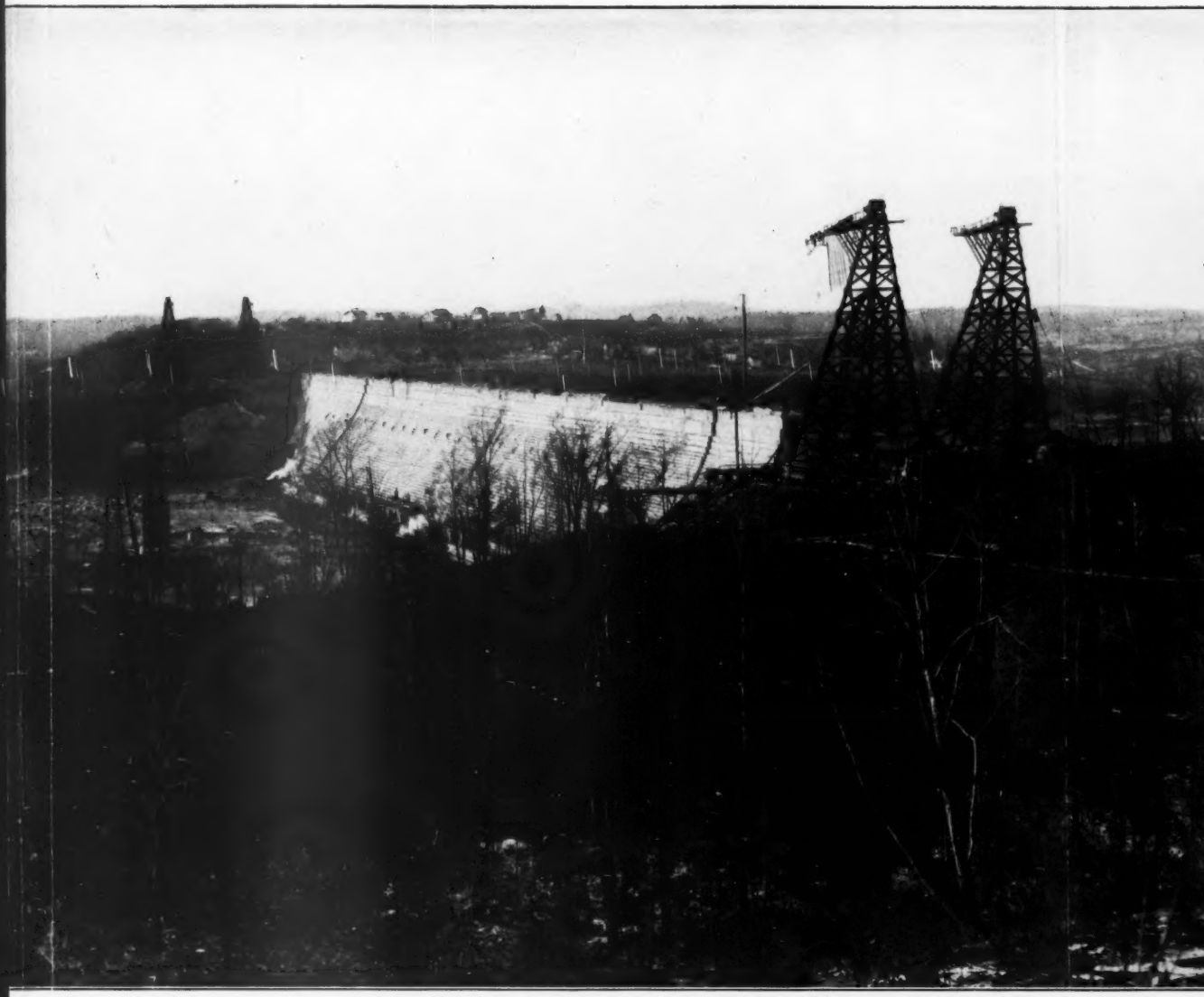

Fig. 9.—View from East Hill on Dec. 23, 1914, at End of Season.

KENSICO DAM

FOR THE CATSKILL WATER-SUPPLY OF THE CITY OF NEW YORK

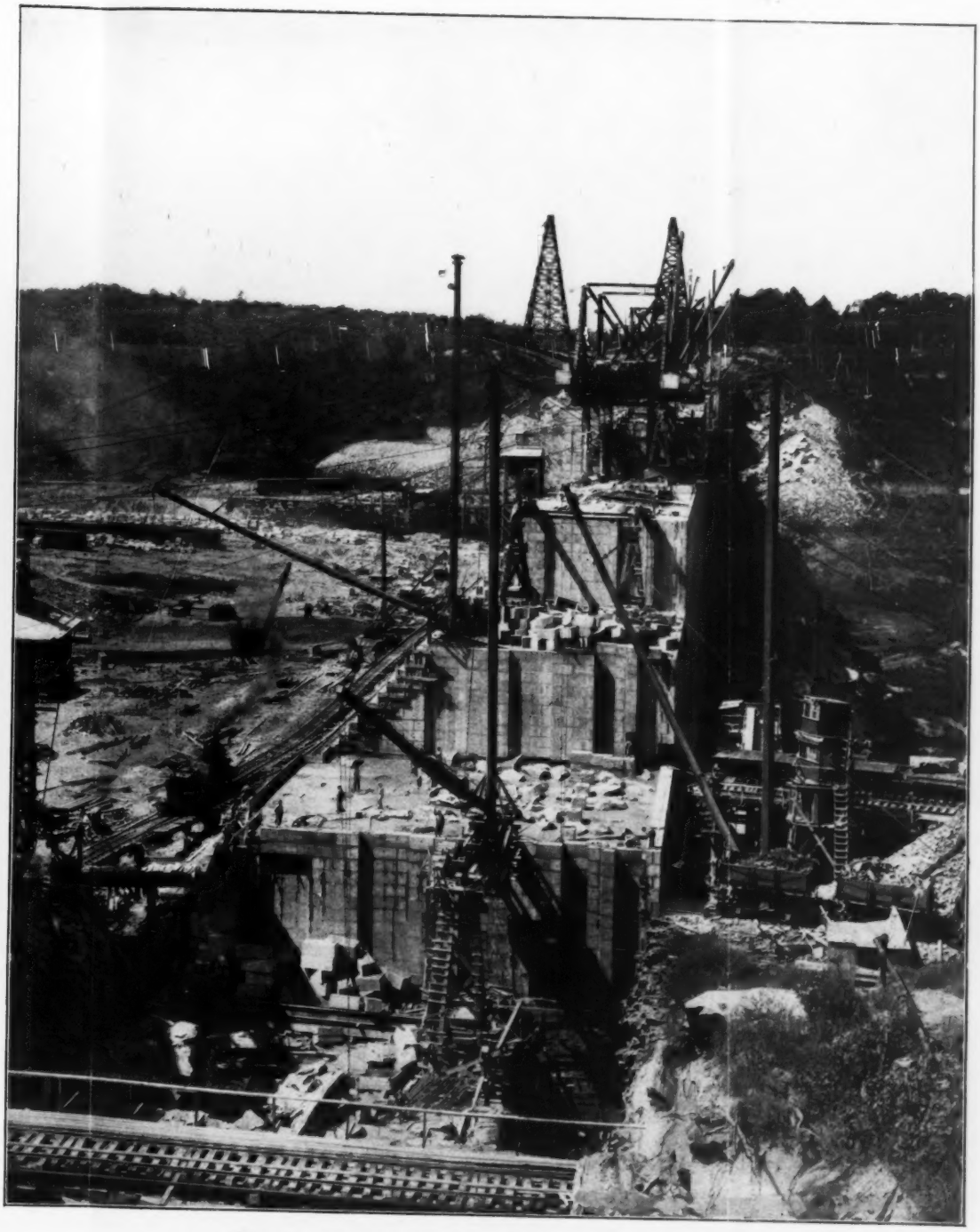

Fig. 8.—View from East Hill on Sept. 9, 1914.

Construction by guy derricks in foreground.
Travelers on seventh lift.

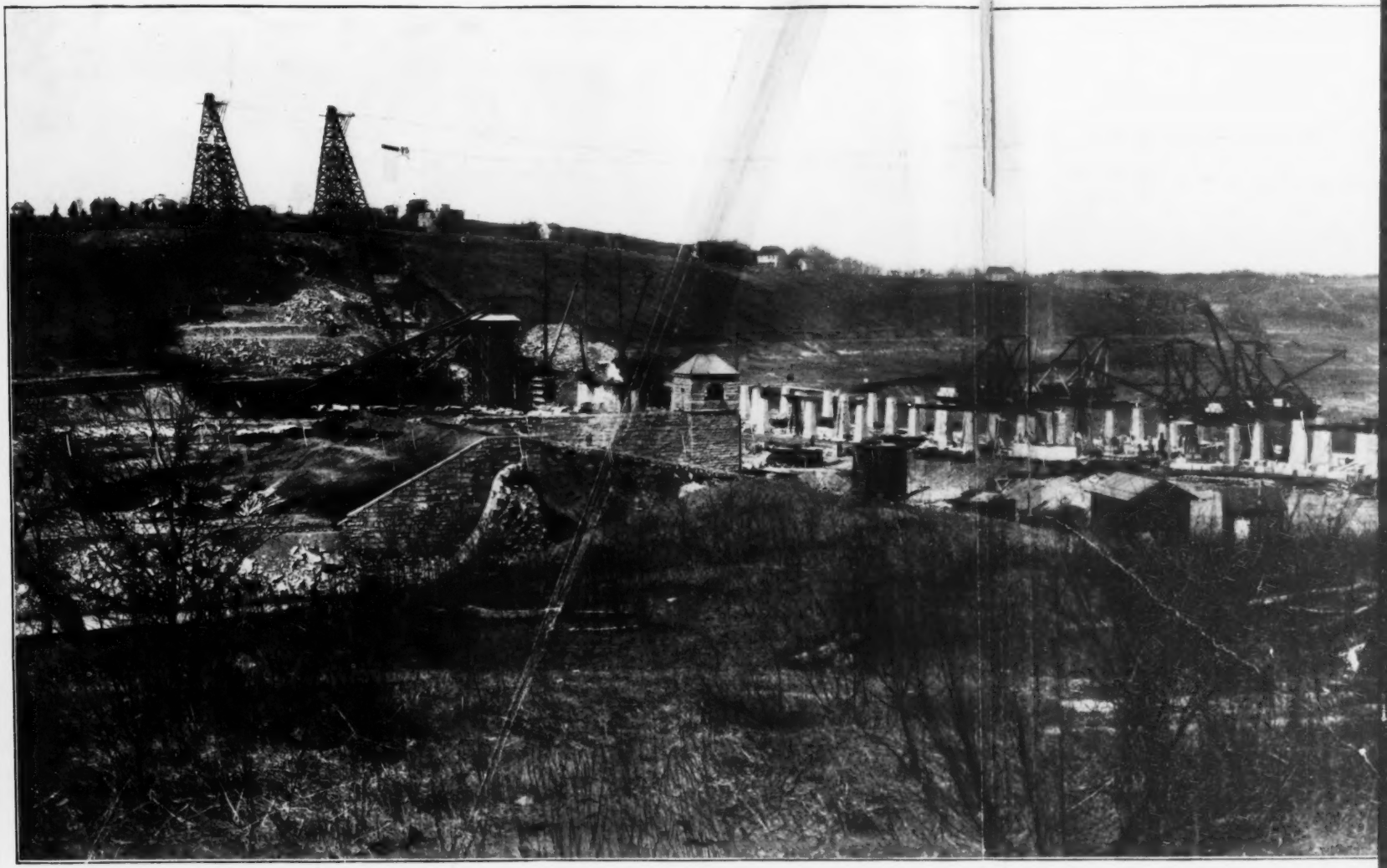

Fig. 6.—Panora
Work is in progr

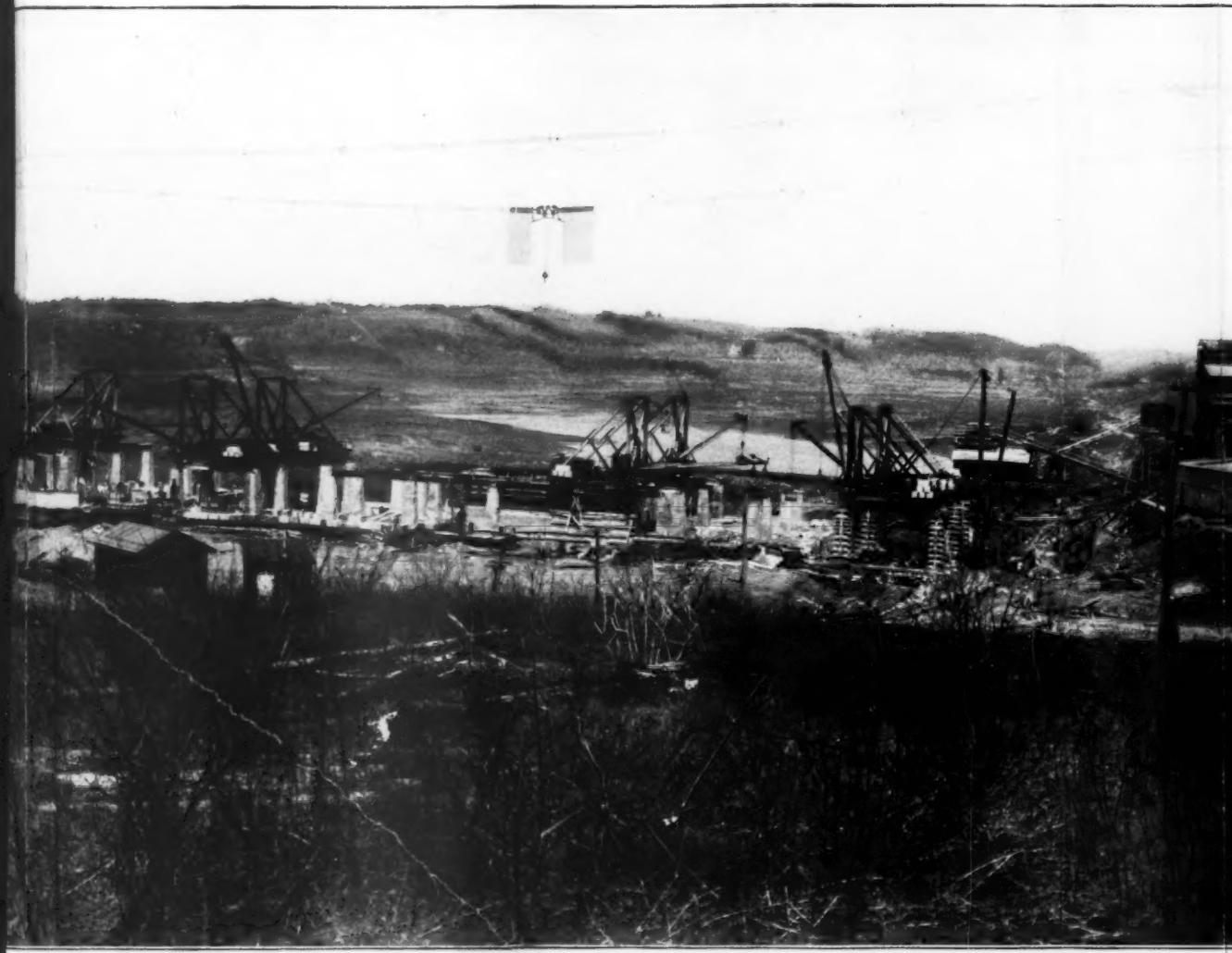
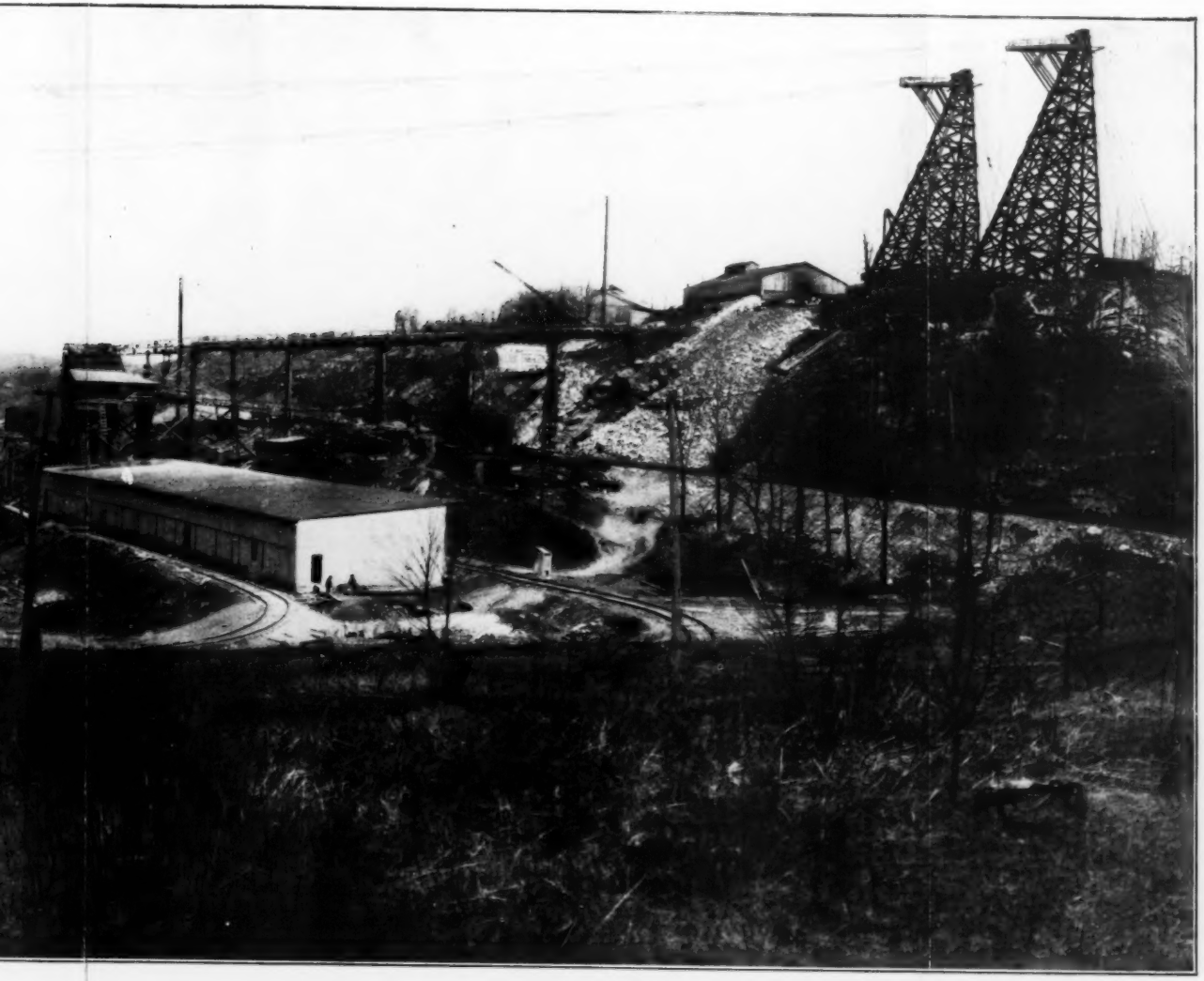
Fig. 6.—Panoramic View Showing Stage of Work in March, 1914.
Work is in progress on the third lift. Concreting plants visible at right and left.